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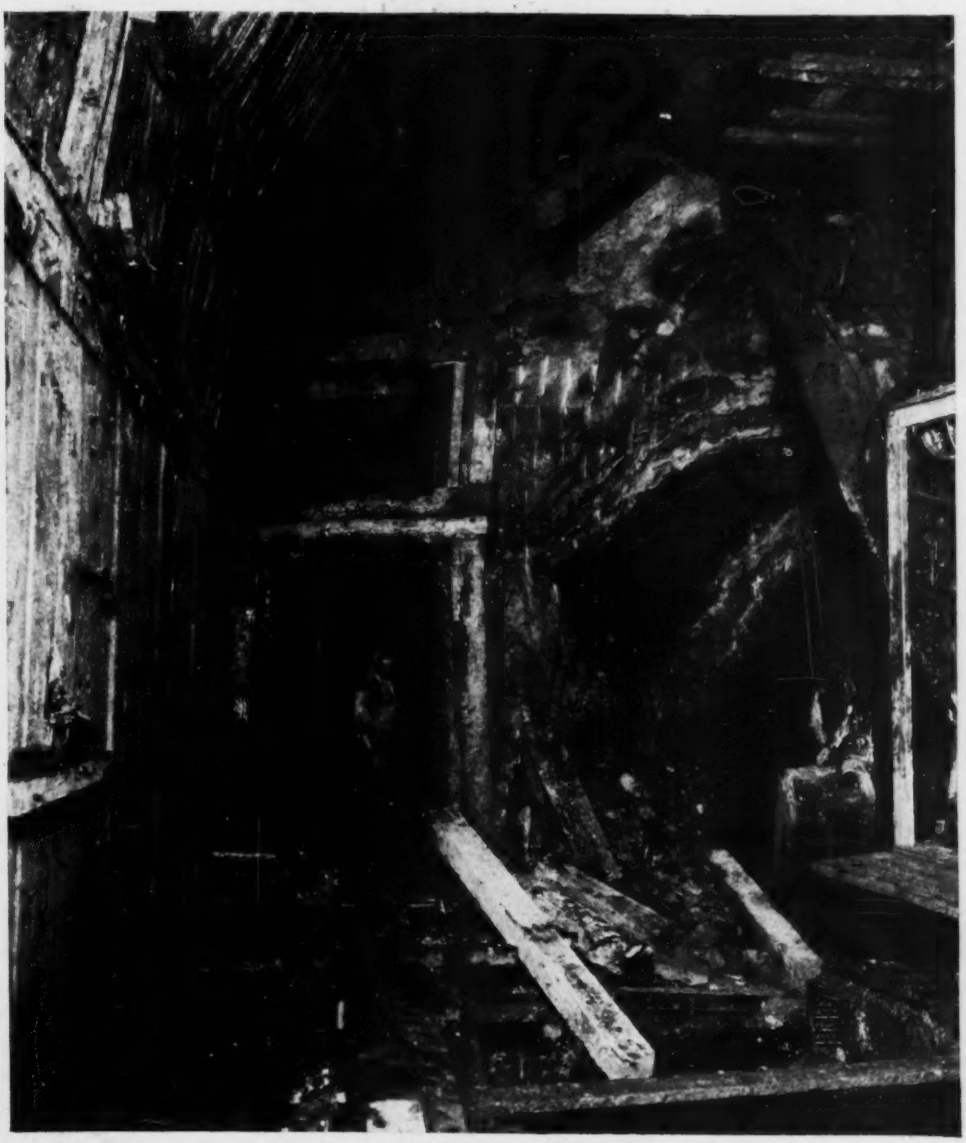

Full-Section Enlargement, Musconetcong Tunnel, New Jersey

Tunneling at Musconetcong Now and Fifty Years Ago

First of Two Articles

Today in the Mississippi Flood Area

Fourth of a Series of Articles

Southern Pacific Railway of Mexico Completed

17-Mile Belt Line Sewer at Springfield, Ill.

Four Pages of Job and Office Articles

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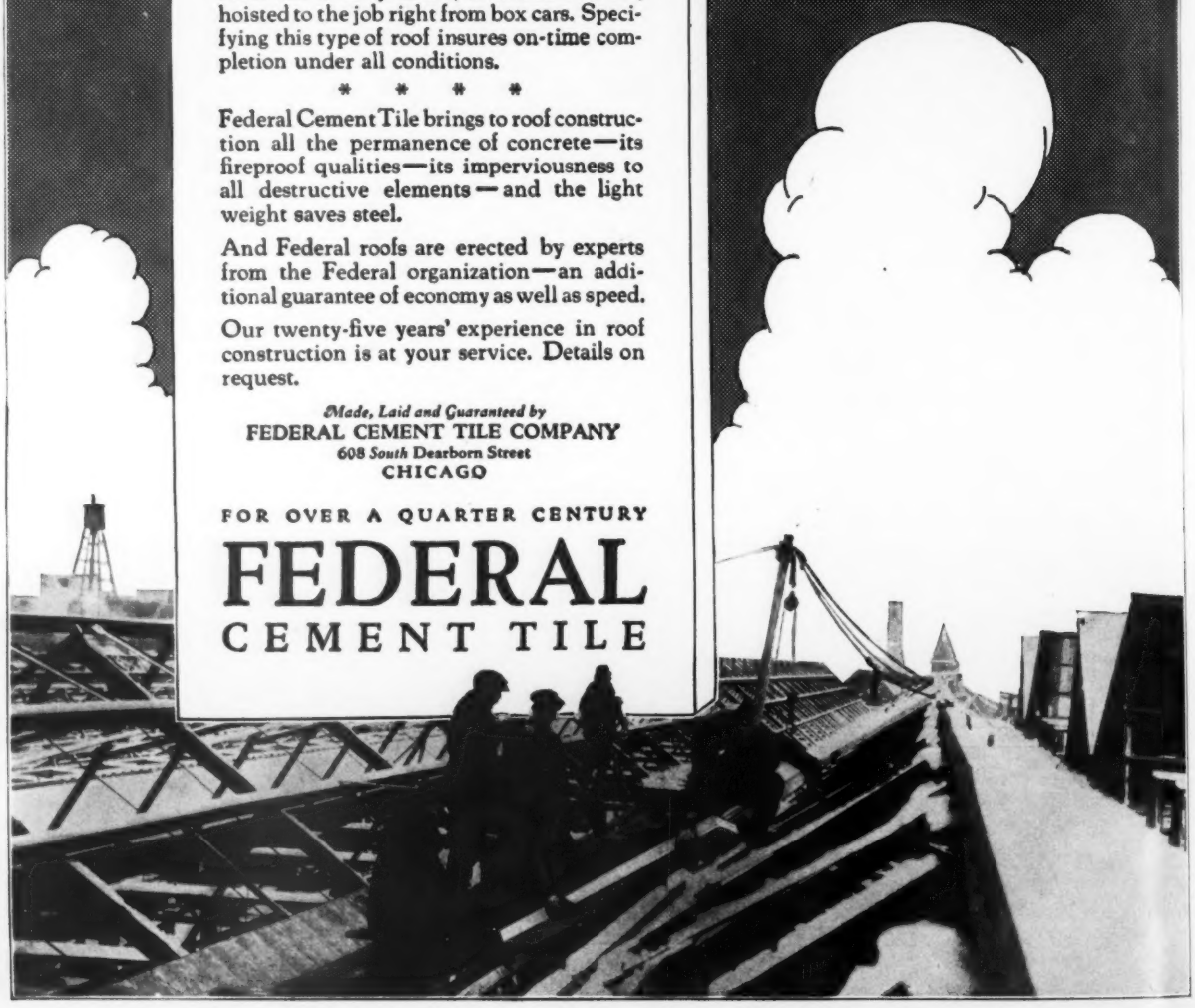

A Consolidation of Engineering News, Engineering Record and The Contractor

ENGINEERING NEWS-RECORD

DEVOTED TO CIVIL ENGINEERING
AND CONTRACTING

McGraw-Hill Publishing Co., Inc.
JAMES H. McGRAW, *President*
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Volume 99 NEW YORK, DECEMBER 22, 1927 Number 25

Another Year

AT THIS season of the year it is fitting to indulge in some introspection as a precedent to clearing the decks for another year. Christmas is rich in tradition and is susceptible of a varied personal interpretation; but its spirit should be inseparable from that of good will. That good will does not mean the temporary dismissal of a grudge, the smile for a less fortunate neighbor, a gift for a friend, or the payment of religious homage. Good will is enduring. It is leavened by a day-to-day tolerance and a seeking after and repetitive expression of what truth civilization has thus far adduced. So, forgetting for the moment personal expressions, let us wish each other a greater disposition toward tolerance, without undue humility, that must make for a wholesome improvement in the art of living.

Many Happy Returns

MOST fittingly the Philadelphia Engineers Club last week celebrated the fiftieth anniversary of its foundation. Such an event is so rare—if memory serves correctly, it is unprecedented—that it is worthy of special observance. Local engineering organization does not often display the basic vitality that is essential to long life. Though its influence upon professional solidarity is transcendent, and the local group should therefore constitute the foundation of engineering community life, it does not often flourish and bring forth real accomplishments. Fortunately, in the city of brotherly love the co-operation of our professional brethren has proved to be more effective and lasting. Its semi-centennial is a brilliant object-lesson of success that may fitly inspire other local engineering groups. We sincerely hope that we may live long enough to enjoy the fifty-year anniversaries of other local societies. And, if extravagant hope is allowable, we will look forward also to the pleasure of meeting with the Philadelphia Club when it rounds out its full hundred. For the present, *Engineering News-Record* wishes it many happy returns.

Anticipating Floods

STEPS are being taken in New York state to anticipate, and if possible prevent, a repetition of the floods which visited part of the state recently, or even the occurrence of a still greater flood such as did so much damage over the border in New England in November. A survey is to be made of the streams of the state to determine what can be done to reduce flood flows; also to find out where reservoirs can be built to hold back excess waters. This proposal is an excellent one, particularly at the present time when the idea of building storage reservoirs at the headwaters of the rivers of the state is not generally favored, due, in large part, to the activities of property owners in the Adirondack Mountains who fear that their holdings will be damaged thereby. Some years ago New York state set up the machinery for building such reservoirs by passing laws authorizing the organization of river regulating districts to finance and build them and presenting a method of assessing costs upon property benefited. Naturally, as the water power interests will benefit most through the construction of storage reservoirs they have been active in agitating for the setting up of river regulating districts. Their activities have resulted in the impression getting abroad that the chief object of the regulating districts is to have the state finance and build reservoirs for the sole benefit of the power companies. Their benefit in reducing floods has been lost sight of. Such a state of affairs is unfortunate. If the floods of the past year serve to impress upon the people of New York state the fact that reservoirs such as the regulating districts are authorized to build may serve a much broader purpose than to increase the profits of the power companies, the floods will not have been without advantage.

Public Money and Private Cupidity

IT IS a sad fact that where the people's money is at stake the individual conscience is apt to be light. Just now more than one instance of this unfortunate moral attitude could be cited, ranging from sewer contract graft allegations in New York to campaigns for free-loan swamp drainage in the Middle West. But the perennial example is furnished by the ever-repeated requests of the federal reclamation landholders for postponement of their contractural repayment of costs to the government. Only a few years ago, after long continued campaigning for such favors, these landholders were gratified by a wholesale write-off of a large part of their obligations. It was thought that this amazing act of generosity would bring their complaints to an end, and assure regular, conscientious payment of future instalments of what was left of the debt. The discussions in Congress made this quite clear. Yet the landholders have not ceased to delay or resist payment, and to petition individually or collectively for postponement of payments or for extension of the repayment time from 20 to 40 years. Within the last year, seven or eight of the federal projects requested the Reclamation Bureau to extend their payment terms. An elaborate investigation was made to determine how far the actual economic difficulties of the projects justified these requests. The findings, just made public, show that, while three of the applications are based on some measure of reason, the others are entirely unjustified and are accordingly to be rejected. It must be understood in this matter of reclamation finances, that the landholders continue to pay their debts to private lenders, although in theory

the government is the preferred creditor and has a prior lien on the land. Government investment in a prior lien, however, is apt to have a dubious status when all the complications of administrative management and congressional control are considered. The reclamation difficulties grow out of the evil of undertaking land development for private benefit through government funds loaned without interest, to be repaid at some variable time in the future. But even beyond this is the underlying evil of deficient moral sense toward public obligations. Only by impartial enforcement of these obligations through the government officials to whom they are entrusted can this evil be checked.

Tunneling Invention

THE contrast in tunneling machinery which is presented in this issue by the article on the Musconetcong tunnels deserves notice. In looking at the old machines and methods set up by the side of those of today, one is bound to inquire how far we have gone in fifty years and of what sort is our advance. The general methods of breaking out the tunnel bore whether in solid rock or in soft ground show no invention in the fifty years. In solid rock the full width top heading and bench method of today is essentially that of 1874 and earlier times. In soft ground the English system used on the old tunnel has, in the new tunnel with its greater pressures, been replaced by a modified European system fully as old in invention. The old contractor was quite as equipment conscious as the present capable organization. He used every resource of machinery then available for use. Indeed, except concrete and the machinery for producing and placing it, there is being used for the new tunnel no machines or supplies that do not have their counterparts in type in the old tunnel operations. The working equipment of today, however, is very much superior in mechanical design, in perfection of workmanship and in efficiency. Indeed compared with the present machines the old drills and compressors, the prime movers which operated them, the explosives and the firing methods, the shovels and hauling equipment, and the mechanical set up for keeping all equipment at the peak of efficiency were small and crude compared with the corresponding features of today's tunneling plant. Altogether the progress evident is pronouncedly the creation of the equipment maker rather than of the construction engineer. This is no reflection on the constructor; his alertness to seek and use more efficient and powerful equipment and better working materials has helped to keep keen the invention of the manufacturer and inventor. This and his higher standards of organizing construction and his greater finesse in using machinery are the constructor's contributions to Musconetcong. The equipment maker outranks him in invention during the period since the first Musconetcong established mechanized tunneling in America.

Queens Borough Allegations

ALLEGATIONS of sewer construction irregularities in Queens Borough, New York, noted in our current news section last week and this, become more serious as the investigations proceed. Should the charges of excessive cost due to collusion, fraud and incompetence extending through a dozen years be sustained, one of the darkest chapters of many years would be added to the history of city misgovernment. A thorough sifting of the whole matter should be made, and doubtless it will be, now that it has been taken up ably and vigorously by the commissioner of accounts of New York City and been referred to a special investigator by Governor Smith, as a result of charges filed with him in a petition for removal of the borough president of Queens. Happily for the engineering profession, no one has yet asserted that any of the alleged excessive sewer costs went into the pockets of engineers; but unfortunately, the charges assert connivance on the part of at least one engineer and imply that a number of others must have kept their eyes, or at least their mouths, closed as to goings on. Doubtless any and all engineers alleged to be in any way responsible for the excessive sewer costs will have full and free opportunity to clear themselves before the taking of testimony is ended.

A Startling Discrepancy

A STARTLING difference between official cost estimates comes to light with publication of the Mississippi River Commission's report on flood control. Its project, differing only in a few particulars from that put forward by Gen. Jadwin, is estimated to cost considerably more than twice as much as the latter—$684,000,000 as against $296,000,000, both figures excluding land cost and damages. The commission, as may be seen from our abstract in this issue, includes a spillway at Caernarvon, but this accounts for only ten millions of the difference, while on the other hand it omits the New Madrid floodway provided in Gen. Jadwin's project. Thus the essential difference in cost of the two plans is wholly without explanation.

The great discrepancy is a vital point of the case. Are the unsettled and uncertain phases of Mississippi flood control so important as to make two equally conscientious estimates differ in the ratio of 300 to 600? Or, are there such essential differences between the plans themselves—differences not revealed in the two reports—that the estimates truly reflect a dissimilarity in scale of magnitude and difficulty of the two projects? Citizens of the United States can hardly contemplate either alternative with equanimity; they will not be ready to authorize an enterprise fully as great as the Panama Canal, and of even more immediate relation to the well-being of many thousands of people, when such extensive vagueness and doubt surrounds the plans.

In another respect the two reports exhibit a startling lack of correspondence. They differ in a number of essential phases of treatment of the river, but they do not explain the difference or even make mention of it. The commission report, being of earlier date, has more obvious reason for this failure than has the Jadwin report. As the case stands, the country has before it two reports, similar but not identical in their technical findings, hopelessly disagreeing in estimate of cost, and entirely unrelated to each other by mutual explanation or discussion.

Despite this confusing condition it is nevertheless urgent that progress be made toward dealing with the great flood problem. Nor ought there be any long delay, for great floods do not always conveniently observe the time spacing which average chance would assign. Protection is needed just as quickly as it is humanly possible to provide it. Congress must not fail to decide upon action within the next two months, so far as effective prosecution of the coming season's work is involved. Though it need not decide upon an ultimate project, it

must direct present action that will be in harmony with what may later be found necessary for full protection. And Congress must also make provision for adequate study of the huge problem, study of such character as to leave no substantial doubts remaining.

Accordingly the hearings on the subject before the Congressional committees are due to take on the subject of thorough investigation, full enough to reveal the nature of the discrepancy between the two projects now reported. That such investigation cannot and should not undertake to question engineering conclusions is obvious. So far as these are involved the only sound course is to submit the questions to an impartial and competent commission of engineers. The discrepancy of costs is one that Congress may well study, however. If the country is forced into the Mississippi project without a clear prevision of the financial scale of the task, a decidedly unpleasant national awakening is inevitable.

Within the coming weeks, then, facts and decisions of nationwide engineering importance will come to the front. They are greater, indeed, than our discussions hitherto have indicated, since the fundamental problems of sound economics involved in the apportionment of cost must be dealt with. But even the engineering and administrative phases themselves are of surpassing complexity, and call upon the wisest foresight at our command. The engineering profession of the whole country rests under the obligation to contribute of its best to the right outcome.

Dams Thin and Massive

IN THE construction of dams, safety controls the planning to a superlative degree. Because the solid dam, with its manifest virtues of bulk and thickness, has been considered the highest embodiment of safety, it has held a dominant place in the field. Its dependability and permanence are almost without question, even in the face of some catastrophal failures arising from errors of design and construction. But of late the modern types which utilize thin concrete sections—on the one hand single-arch dams and on the other hollow dams, whether buttress-and-slab or buttress-and-arch—have assumed an increasingly strong position, and now they imperatively claim attention. They usually have a cost advantage over solid dams of either stone or concrete, and their advocates claim for them equal safety. If experience substantiates this claim against all agencies of attack and deterioration, the cost factor cannot be ignored, the more so as there are many projects which could not be carried out if only the massive type of dam could be considered.

There is already a considerable amount of experience with thin dams, and their favorable showing is strengthened by what has been learned in recent times about better concrete-making. An indication of the widespread interest in the subject appears in the fact that no less than 86 organizations and individuals contributed to the fund for the Stevenson Creek test dam experiments. It is also worthy of note that a new type of thin dam, the multiple-dome form, is now being built on the Gila River in Arizona. On the other hand, in Europe a distinct trend of engineering sentiment against thin dams has developed, doubtless in large part because of the failure of the Gleno dam, which was of the buttress type, though its construction was due to the giving way of its massive foundation masonry structure.

Inherently the thin dam is entirely safe; there is no weakness hidden in the type itself. Its margin of security is attested by two high multiple-arch dams in southern California—one with a height of 150 ft.—that continue in satisfactory service despite extensive cracks in their buttresses.

The inherent safety of the type conceded, there come then certain considerations that may not be overlooked, particularly those that bear on stability and permanence. It is these conditions which, upon full appraisal, with the help of a careful study of experience, will determine the ultimate relative position of massive and thin dams from the safety viewpoint.

Thin dams of the buttress type depend partly on steel reinforcement to carry the load. Usually, also, they subject their material to higher stresses than are commonly found necessary or desirable in massive dams. These two elements in themselves place more severe demands on construction procedure than does the older type. The positioning of the steel must be accurate and the quality of the concrete must be such as to withstand the higher stresses in an enduring manner if the structure is to be given a permanence rating on a part with massive dams.

Equally important is the matter of tightness. The danger of leakage must be carefully avoided. Percolation of water through the structure is the sure forerunner of decay. Accordingly, meticulous control of concrete proportioning, mixture and placing is necessary if the finished structure is to realize the enduring life postulated in its design. Again, troubles that may result from freezing on the upstream face have to be considered, together with other causes of damage to concrete arising from temperature variations in thin concrete structures.

In the matter of emergencies it must be remembered that thin dams are more tempting to persons who might seek their destruction. Slight tectonic movements, also, are more likely to cause cracking. A major earthquake would probably have more serious effect on a thin dam than on a massive block of masonry solidly filling the valley. But in this connection it is proper to remember that the buttressed dam is quite as much a gravity structure as is the massive dam. The buttresses are similar to gravity sections, separated by gaps spanned by the slabs or arches. If buttresses were made sufficiently heavy, the type might well be considered secure against major disaster even though a single span of the slab or arch might fail due to earthquake or other causes.

The further study of various types of thin dams, and the present trend toward their more extensive use where conditions are favorable, are distinctly progressive moves. The state of the art of hydraulic development in the United States is clearly conducive to further progress in the field of this type of structure. The thin dam is today definitely beyond the stage of experiment. But confidence in these structures and hope for their more extended use must clearly be joined with thorough appreciation of the fact that in thin dams more than in massive structures the rôle of quality in both design and construction is particularly vital. There may at no time be anything savoring of flimsy design, special study must be given to the selection of materials, and the execution must at all times be under competent supervision and unusually strict inspection. If the possibility of mischance in these phases of the undertaking be given weight, it will doubtless be necessary under present-day conditions to continue to rate the massive dam superior to the thin dam in its assurance of long-time safety. But the art is progressing so rapidly that it may be only a short span of years before this difference is eliminated or even reversed.

FIRST OF TWO ARTICLES—

Tunneling at Musconetcong Now and Fifty Years Ago

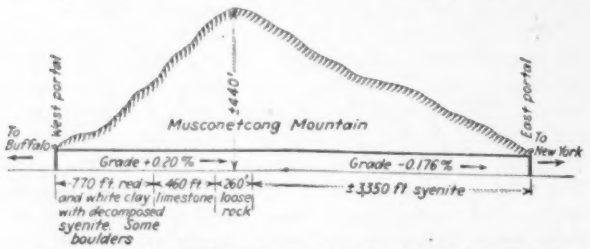
Tunnel Construction Plant of 1874 Contrasted with Plant Employed on Present Tunnel Nearly Identical in Location, Length and Materials Penetrated

A notable contrast in tunneling methods and machinery is furnished by the description given in this and our next issue of parallel tunnels through Musconetcong Mountain in New Jersey. One tunnel was begun in 1872 and was completed in 1875; the other was begun in 1926, is now under construction, and will be completed in 1928. On the same railway, scarcely two rods apart, through identical rock strata, of almost equal length, but separated 54 years in date of construction, these tunnels enable comparisons rarely afforded by such engineering work.

Virtually, mechanized tunneling methods first became accepted practice at old Musconetcong. In the new tunnel mechanized tunneling in its most advanced manifestations is exemplified. The change in half a century is of curious interest in that it demonstrates high enterprise and skill in the perfection of tunneling tools and materials rather than striking advance in rock tunnel construction processes in that time.

As the story is largely one of equipment development, illustration has been employed profusely both to emphasize the broad fact of increased mechanization of tunneling and to enable comparisons of the machines of today with those in use half a century ago.—EDITOR.

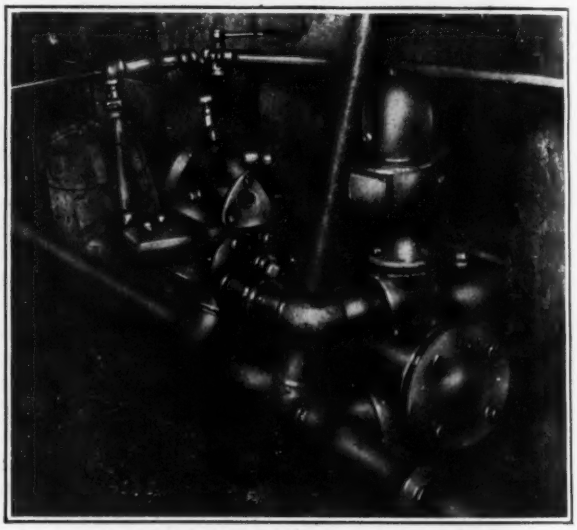
OLD MUSCONETCONG tunnel was built in 1872-5 to give the Lehigh Valley R.R., then terminating at Phillipsburg, N. J., on the Delaware River opposite Easton, Pa., an exit to tidewater through Musconetcong Mountain, one of the high ridges of western New Jersey. This new tidewater extension, built as the Easton & Amboy R.R., ran to Perth Amboy on Raritan Bay. The tunnel was the outstanding construction task. It was 4,829 ft. long, with a rock section 21 ft. high and 26 ft. wide, for two tracks. With growing car and locomotive sizes, the tunnel finally could not pass two trains and was changed to single track. Meanwhile, the railway to the tunnel both ways had grown to four tracks. Obviously, the constriction of the mile of single track had to be reduced and in time a new Musconetcong tunnel was decided on—a two-track tunnel 30x25 ft. in section and 4,840 ft. long, parallel to and 130 ft. south of the old tunnel. This is the tunnel now being constructed. Compared with the old tunnel, the greater cross-section is the only broad difference in general engineering design introduced to meet modern traffic requirements.



PROFILE OF MUSCONETCONG MOUNTAIN SHOWING GROUND PENETRATED BY NEW TUNNEL

Virtually the same strata, except that at the west end they contained great volumes of water, were penetrated by the tunnel constructed 1872-5.

Location and Materials—The diagram profile of the new tunnel indicates the materials penetrated. The ground penetrated by the old tunnel developed great volumes of water, whereas the new tunnel workings, in identical material, are comparatively dry. The only water trouble to date was caused by striking a flow of about 300 gal. per minute in the north plumb-post drift on the west end at the junction of soft ground with limestone, which flooded the workings, causing two days' delay. The flow is now about 200 gal. per minute, and is handled by the continuous operation of pumps. The syenite is a hard rock but blocky enough in places to require timbering to guard against roof falls. Where the syenite joins the limestone, the rock is loose and has to be heavily timbered. The limestone itself is reasonably stable but shows many cavities. From the solid limestone west to the portal, the mixture of clays and decomposed syenite is very difficult material to tunnel; it develops enormous pressures and is highly unstable when wet. It has been in this material and in the zone of loose rock that virtually all of the heavy work in the new tunnel has been encountered.



AIR-OPERATED PISTON PUMPS FOR DEWATERING NEW TUNNEL

Steam-operated Knowles and Niagara pumps were used in numbers in the shafts and slopes of the old tunnel.

In the old tunnel the cavernous limestone held water pent up in numerous cavities. This water gave much trouble, as is described farther on. The old tunnel, how-

AIR-OPERATED HOIST LIFTING CARS TO CORE TOP FOR CROWN DRIFT

There was in the old tunnel no air-operated machinery except the rock drills; compressed air in 1874 was a new agent in tunnel construction.

ever, drained the rock so that when the new tunnel came to the limestone it was, except as stated, comparatively dry. Indeed, water gave no serious trouble in the new tunnel, nor did it in the old, once the limestone reservoirs had been drained. This drainage, however, proved a nerve-breaking task in the old work.

Access, Power Supply and Plant—The new Musconetcong tunnel has a four-track railway coming to each end of the work. Access for materials, supplies and equipment, therefore, was no problem beyond siding and spur construction and some trucking of cement and camp provisions and for the interchange of supplies between ends. When the old tunnel was begun there was no railway nearer than a mile from the west portal and five miles by over-mountain roads from the east portal. All materials and supplies had to be hauled these distances by teams; this hauling kept busy for nearly three years 24 four-horse teams. The comparatively little hauling by highway for the new tunnel is by motor trucks; and the men come to work, go to town and cross the mountain from east camp to west camp, by automobiles. This has been the change in highway haulage between old Musconetcong and new Musconetcong.

The new power is quite as revolutionary. At the old tunnel, steam operated every prime mover. It is stated that 1,000 tons of soft coal and 26,500 tons of anthracite coal were used for steam generation and blacksmithing operations. On the new work the prime movers are electric motors. A 7-mile transmission line brings in 33,000 volts from the New Jersey Power & Light Co., and transformers at the west portal step this current down to 2,300, 220 and 110 volts. All equipment except the steam shovels for approach cut excavation is operated by electric motors or by compressed air generated by motor-driven compressors. The steam shovels, when shifted into the tunnel for mucking, were changed to compressed air. Pumps and hoists in the tunnel are air-operated. A 200-hp. motor operates each of the four compressors. Air is used to blow the oil forges and operate the drill sharpeners. Electricity lights camp and plant buildings and the tunnel workings; it drives the 4,000-cu.ft.-per-minute blowers which force fresh air through 18-in. pipes to the working faces. These forms and applications of power are not notable from the standpoint of present

PLANT ITEMS AT OLD AND NEW MUSCONETCONG TUNNELS

Only main items are scheduled, without indicating capacity or output, merely to illustrate mechanization.

Old Musconetcong 1872-5	New Musconetcong 1926-8
26 Ingersoll drills	30 Ingersoll-Rand drills
4 Burleigh and 4 Rand-Waring air compressors, steam-driven	4 Chicago Pneumatic Tool Co. compressors, belt-driven by electric motors.
13 locomotive boilers supplying steam to all plant except steam shovels	7-mile transmission line carrying 33,000 volts; power for all plant except excavators
2 steam shovels for west approach cut	2 steam shovels and 1 dragline for approach cuts and mucking
2 locomotives handling spoil trains	8 locomotives handling spoil and concrete trains
	2 concrete mixing plants
	2 pneumatic concrete placers
	Drill sharpeners, compressor after-coolers and ventilating plants.
Hoisting engines, pumps, derricks, repair shops, job and camp structures and miscellaneous tools	Hoists, pumps, derricks, shops, camp and miscellaneous tools and outfits.

practice, but are named merely in comparison with the power equipment of some dozen or more steam boilers employed in the old tunnel workings.

The comparison of tunnel construction equipment exhibits greater similarity. Compressed-air drilling and high explosives were first used in tunneling in America on the Hoosac tunnel but only in part of the work. Liquid nitro-glycerin was the explosive. The Nesquehoning tunnel in Pennsylvania was the first to employ compressed-air drilling throughout, but the blasting was done with black powder. At Musconetcong there was complete mechanization of drilling operations and dynamite was the explosive, its first regular use, it is believed, for tunneling in America. Indeed, power equipment was used generally for the first time at old Musconetcong—it was the pioneer in modern rock-tunnel planting practice. Writing in 1875, Henry S. Drinker, then resident engineer on the tunnel and later author of the great American classic on tunneling, said of the construction

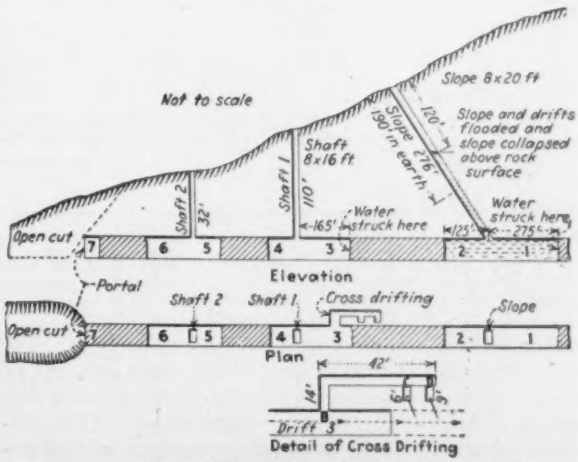
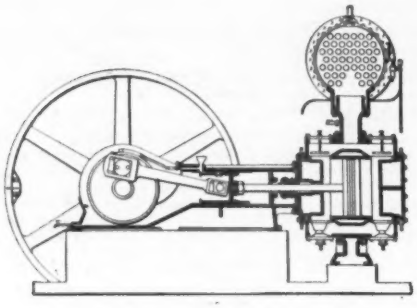


DIAGRAM OF CROWN DRIFT OPERATIONS IN OLD TUNNEL

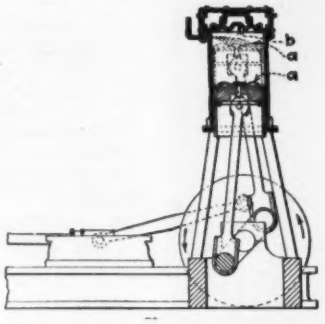
These complex workings were resorted to for handling the water which repeatedly flooded them and ultimately delayed construction about twelve months.

plant: "The contractor had in use the latest and most approved machinery." A comparison of main plant items then and now is made in the accompanying table. Aside from differences in capacity and perfection of machines, the outstanding differences are electric operation and the concrete mixing and placing equipment. The particulars of comparative equipment are given in succeeding sections, working from rather scanty records of the old machines found in contemporaneous publications.



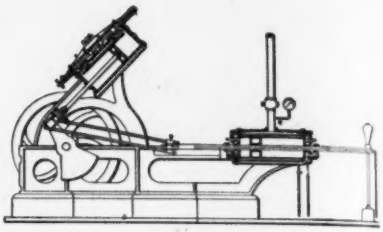
COMPRESSOR BELT-DRIVEN FROM 200-HP. ELECTRIC MOTOR

Four of these machines, two at the east and two at the west end of the new tunnel, supplied air for drills, hoists, pumps, power shovels and drill-sharpening shops.



BURLEIGH COMPRESSOR AT WEST END OF OLD TUNNEL

Four of these compressors, each rated as a two-drill machine, were used at the west end. Each was operated by a horizontal engine and a 45-hp. tubular boiler.

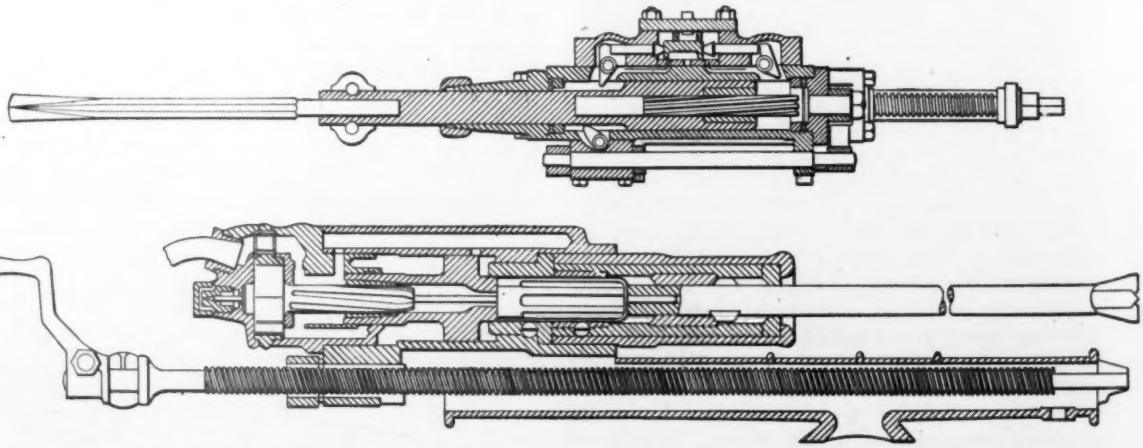


RAND-WARING COMPRESSOR AT EAST END OF OLD TUNNEL

Four of these compressors with five 50-hp. locomotive boilers supplied air for the east-end workings. Note the oscillating steam cylinder with piston rod direct-connected to crank arm.

Approach Cuts and Start of Headings—About 370,000 cu.yd. of material had to be taken out for the approach cuts for the new tunnel. These cuts were about 75 ft. deep at the portals, the longer cut being at the west end. The records at hand of the old tunnel cuttings do not give quantities but probably these were not very much less than those for the new tunnel. Depths and materials of excavation were certainly about the same. The interesting fact is that in general the same methods of excavation were employed in the two operations over half a century apart—steam shovel excavation and train disposal. Accounts of the old work are meager in information concerning the approach cuttings but on the west approach two steam shovels "ran day and night" and with "two small locomotives" took out the cut.

In 1872-5 Souther was the only maker of steam shovels and the Otis shovel was the only type, so these were probably the shovels used on the old tunnel. Even without particulars it is conclusive that they were puny machines compared with the railroad type steam shovels used, one at each end, on the new tunnel. These were modern railroad type shovels, both loading into 5-cu-yd. dump cars hauled by 20-ton and 18-ton dinkeys and 16-ton gasoline locomotives. The dump car tracks were 3-ft. gage. Gasoline locomotives and dinkeys all are of a day since the old tunnel was built. Only the roughest of performance comparisons is possible. On the west approach the two shovels on the old work made the cut to crown heading level in 19 months; the 70C shovel on the new work had got down to the crown heading in five months.



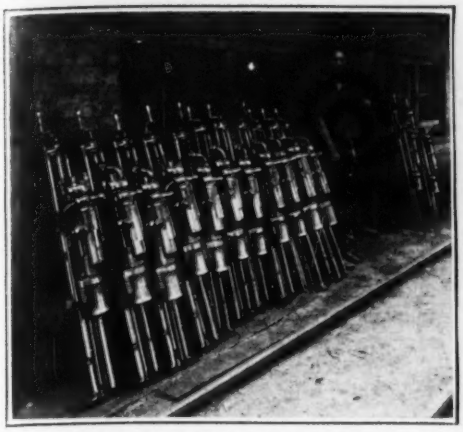
ABOVE—INGERSOLL 5-IN. DRILL USED ON OLD TUNNEL

This type of drill was selected as being less cumbersome than the Burleigh drills used at Hoosac and Nesquehoning. It was its first use in tunnel work. This type of piston drill, improved in workmanship and design, continued in use until the hammer drill, after 1896, gradually replaced it.

BELOW—DRIFTER DRILL USED ON NEW TUNNEL

This drill, 30 of which were used during the construction of the new tunnel is eight times as fast as the first Ingersoll drills and can be shifted in half the time. It is the highest development as of this date by its makers of the drifter drill.

As it performed the important function of making the new tunnel a dry tunnel the west crown-heading and drainage work on the old tunnel calls for summary. With the long deep cutting, the engineer of the old tunnel thought to gain time by driving a slope to the tunnel and drifting both ways from it. This slope was inclined east to strike the tunnel about the middle of the limestone zone. The accompanying sketch serves to explain it and



EAST PORTAL DRILL AND BLACKSMITH SHOP ON NEW TUNNEL

On the old tunnel, says an account published in 1875, drills "were found to run for months with little or no repairing necessary." On the new tunnel all drills undergo regular conditioning.

DRILL EQUIPMENT TRAIN ON NEW TUNNEL OPERATED BY ELECTRIC LOCOMOTIVES

Nippers were important chaps in the old tunnel—every drill runner had one to fetch and carry his "steel." At new Musconetcong drill servicing between shop and face is by train.

the subsequent drainage operations. The slope was begun in April, 1872, and work proceeded well until drift 1 to the east had advanced 275 ft., and drift 2 west some 125 ft. Then on May 7, 1873, water under great pressure was struck in drift 1 and, against all the pumps available, flooded drifts and slope to within 120 ft. of the top. Increased pumping failed to lower the water much and on May 14 the slope timbering above the rock surface began to collapse and the slope in earth 190 ft. down was lost.

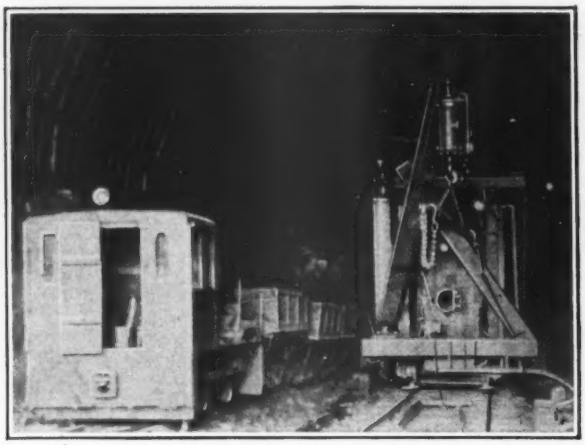
To meet this situation shaft 1 was started and reached grade, 110 ft. down, on June 21, 1873. Drifting began east and west from the shaft bottom and on Aug. 13, when drift 3 was 165 ft. east, water broke in and flooded the workings to a height of 40 ft. in the shaft. No material change in water level in the slope followed, indicating that the second inflow was from a new source. Shaft 1 was pumped out on Aug. 19, on Aug. 22 and on Sept. 9, each time followed by flooding as soon as any advance of drift 3 was attempted. Each time, however, this drift would dam up with sand, so that the workings could be laid dry. Work was then begun on shaft 2 and drift 5 on Sept. 13, and on Oct. 7 drifts 4 and 5 were connected. This gave two shafts from which to pump.

Work was then resumed on drift 3, but it again flooded and was pumped out Oct. 14. This time the dam was left and cross-cutting and parallel drifting were begun to tap the water ahead of the face of drift 3. At cross-cut *C* no water was struck and another attempt was made to advance drift 3 on Nov. 4. Again water came in, but it was dammed before the works were flooded. Then parallel drifting was resumed and at cross-cut *D* the drill hole tapped the reservoir on Nov. 8. After three days' pumping the pumps broke down and the drifts were flooded. By this time, Nov. 11, 1873, the open cut had got down to crown drift level and drift 7 was started and connected with drift 6 on Nov. 20. With free vent the water spent itself in 40 minutes, running drift full. Thereafter the water in both the slope workings and the shaft workings was ordinarily no more than could be handled by pumps and ditches.

As stated, the old tunnel workings laid the mountain dry and the new tunnel has had little water trouble. The crown drift on the new tunnel, however, started

DRILL SHARPENER AND OIL FURNACE AT WEST PORTAL OF NEW TUNNEL

At old Musconetcong drill sharpening was wholly a hand blacksmithing task with bellows and a soft coal forge. Now an air oil furnace will heat from 150 to 300 bits an hour and a pneumatic sharpener will sharpen from 50 to 100 an hour.



TIMBERING AND MUCKING ROCK IN NEW TUNNEL
A railroad-type steam shovel operated by air and a rear-end air hoist load and shift cars in muck trains operated by gas and electric locomotives. This is in the new tunnel. In the old work the cars were loaded by hand and taken to the portal by man or animal power.

from the portals, as faster cutting by the big steam shovels laid the portals clear sooner than slopes or shafts could be put down. Cutting started May 1, 1926, and crown drifting on Sept. 1. These dates were the same at both ends.

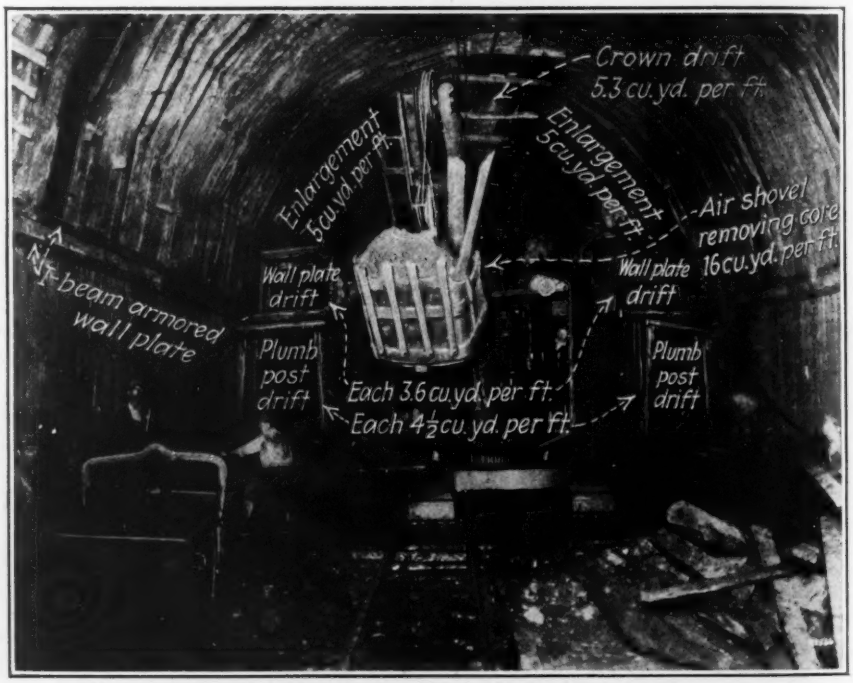
Compressor Equipment—At the new tunnel identical compressed air plants are installed at both ends. Each consists of two 19x11x12-in. duplex compressors belt-driven each by a 200-hp. synchronous motor. All compressed air is run through 3,000-cu.ft. after-coolers with cooling water furnished by electric-driven centrifugal pumps.

A sectional elevation of one of the compressors is given in comparison with similar drawings from prints in contemporaneous publications of the old compressors. The Burleigh compressor, developed largely on the Hoosac tunnel, was used at Nesquehoning and then in the form illustrated at Musconetcong. It was known as a two-drill machine; the four compressors at the west end at Musconetcong actually operated nine drills of the type described later. The compressor, as shown, had two vertical air cylinders whose pistons were actuated from a common crankshaft operated by a horizontal engine and a 45-hp. tubular boiler. The single-action air cylinders, the large circular plate valves *a* and *b*, and the cooling water jet are to be noted.

The other type compressor, known as the Rand-Waring, used at the east end had been used at Nesquehoning; they were bought second-hand for the work at Musconetcong. One had a 16-in. cylinder and three had 12-in. cylinders, double-acting, with poppet valves and a water-cooling system. An oscillating steam cylinder operated the air piston through a crankshaft, connecting rod and crosshead. Five 50-hp. locomotive boilers supplied steam for the four compressors.

Besides their contrast in compactness, the compressors of 1927 are different machines in almost every compressor embodiment except the basic materials of construction and the compressing element of a power-driven double-acting cylinder and piston. Automatic cooling, regulation and lubrication common in modern compressors are new since the Burleigh and the Rand-Waring machines. Adjustability and replaceability of bearings and crossheads, now common, are new. The enclosed type of main frame is new. Bearings and guides are larger; pistons are lighter and are packed to take up wear; cylinders are reborable; simple and light plate valves, with cushioned operation, have replaced poppet and Corliss types; cast steel replaces cast iron in crank disks; babbitt and bronze bushings, forced joints, forged and cold-rolled rods, ground and polished guides, and all materials and workmanship demonstrate improvement.

The motor end shows even greater changes. Then the steam engine was the only available prime mover. Now there are also the internal combustion engine and the electric motor. Quite as notable is the change in simplicity of power transmission from the motor to the compressor element. Finally the oscillating cylinder steam engine of the old Rand-Waring mixer is today as far out of consideration as the single acting compressor cylinder of the old Burleigh compressor. In the Rand-Waring compressor there appears, however, a forethought of today's objective of motor and compressor unit compactness.



POWER SHOVEL EXCAVATING SOFT GROUND CORE, NEW TUNNEL
This view shows the drifts, the core and the timbering where steel inner frames were not necessary. The shovel is air-operated.

[*The second article, completing the description of the work on the old and new Musconetcong tunnels, will be published in an early issue.*—EDITOR.]

Bridge Underclearances Required By Ocean Vessel Mast Heights

A Tabulation of the Mast Heights and Drafts of a Representative List, Including the Largest, of Ocean Vessels

By P. K. Schuyler
U. S. Bureau of Public Roads,
Washington, D. C.

THERE ARE at the present time, proposed and under consideration, bridges across the entrance to some of the major ports of the United States. It is of the utmost importance that such bridges be so planned and constructed that they will not interfere with the entrance of vessels to the port. Restricted clearances will cause shipping to seek other ports and, in time of war, would hinder the movement of supply and naval vessels.

In determining whether bridge plans submitted to the War Department should be approved or disapproved, the department authorities must consider, among other things, the mast height of vessels that can reasonably be expected to pass under the proposed structure. It is, of course, equally necessary for the bridge designer to recognize the same problem. This article has been written with the idea of presenting data which will be of assistance in solving this question.

Most of the larger vessels built in the United States have their masts so constructed that the topmasts can be housed; that is, the upper portion can be lowered. The housing of a topmast takes from one-half hour to one-half day, depending upon the amount of rigging attached to it. Vessels of the U. S. Navy and the American-built vessels of the Emergency Fleet Corporation are constructed so that their topmasts can be housed to such a height as to allow them to pass under Brooklyn Bridge, which has a clearance of 127 ft. 8½ in. at mean high tide. Foreign builders of ships do not consider it necessary to construct them to pass under Brooklyn Bridge, and foreign ship clearance heights run up to a maximum at present of 221 ft., which is the height of the "Majestic." This vessel, as is the general practice with the larger foreign-built liners, has a steel mast that does not house.

In the consideration of the bridge clearance required at a certain port not only must the necessary clearance be

MAST HEIGHTS AND DRAFTS OF OCEAN VESSELS

(Height of Mast Given Is Highest Point of Topmast Above Light Line)

Name of Vessel	Tonnage	Length	Beam	Draft Loaded	Height of Mast, Ft.	Owner
Berengaria	52,226	883	98	39	217	Cunard S. S. Co.
Majestic	56,551	915	100	39	221	White Star Line
Aquitania	45,647	869	97	36'-2"	185	Cunard S. S. Co.
Leviathan	59,957	907	100	39'-6"	215	United States Lines
Olympic	46,439	852	93	34	203	White Star Line
Ohio	18,940	591	72	31	190	Royal Mail S. S. Co.
Asturias	22,137	630	78	30	188	R. M. S. P. Meat Trans.
Empress of Australia*	21,861	590	75	29	184	Canadian Pac. S. S. Co.
Mauretania	30,696	762	88	36'-3"	182	Cunard S. S. Co.
Duilio	23,226	602	76	30	180	Italian-Am. Ship.Corp.
Empress of Canada*	21,617	627	78	29	178	Canadian Pac. S. S. Co.
Belgenland	27,152	697	78	36	175	International Nav. Co.
Homeric	34,351	751	83	36	174	White Star Line
Adriatic	24,541	709	76	37	173	White Star Line
Colombo	12,087	518	64	29	173	Italian-Am. Ship. Corp.
Laconia	19,680	600	74	32	171	Cunard S. S. Co.
Minnetonka	21,998	601	80	36'-9"	170	Atlantic Trans. Co.
Minnewaska	21,716	601	80	37	170	Atlantic Trans. Co.
Orca*	16,063	550	67	33	170	Royal Mail Steam Packet Co.
Orduna	15,499	550	67	33	170	Royal Mail Steam Packet Co.
Mount Vernon*	18,372	685	72	32	168	United States Lines
Agamemnon	18,372	685	72	32	168	United States Lines
Orama	20,000	632	75	29	165	Oriental Steam Nav. Co.
Empress of France	18,357	571	72	29	165	Canadian Pac. S. S. Co.
Gripsholm	17,300	553	74	29	165	Aktiebolaget Goesverkur
Giulio Cesare	21,657	602	77	30	165	Navigazional Generale Italiane
Altic	21,026	681	75	36	164	White Star Line
Arabic	16,786	590	69	30	163	White Star Line
Empress of Scotland	25,128	677	77	34	162	Canadian Pac. Lines
George Washington	23,788	699	78	33'-5"	162	United States Lines
Reliance	16,798	592	72	28	162	Hamburg-American Line
Resolute*	17,258	596	72	28'-9"	162	Hamburg-American Line
Albert Ballin	20,815	602	79	33	158	Hamburg-American Line
America	21,144	669	74	33	157	United States Lines
Montroyal	15,646	549	66	31	156	Canadian Pac. S. S. Co.
Champollion	12,263	495	62	25	156	Sec. des Services Cont.
Carinthia	20,277	600	73	32'-3"	155	Cunard S. S. Co.
Monte Sarmiento	10,876	500	65	29'-6"	155	Hamburg S. America SS. Co.
Franconia	20,175	601	73	31'-4"	155	Cunard S. S. Co.
Melita	14,000	520	67	28	153	White Star Dominion Line
Minnedosa	14,000	520	67	23	153	White Star Dominion Line
Aurania	13,900	519	65	31	153	Cunard S. S. Co.
Antonia	13,867	520	65	30	153	Cunard S. S. Co.
City of Los Angeles*	12,550	560	62	29'-6"	152	Los Angeles S. S. Co.
Montcalm	16,413	549	70	28	150	Canadian Pac. S. S. Co
Montrose	16,402	549	70	28	150	Canadian Pac. S. S. Co.
Montclare	16,314	549	70	28	150	Canadian Pac. S. S. Co.
Arcadian	12,015	520	62	28	150	Royal Mail Steam Packet Co.
Araguaya	10,530	515	61	27	150	Royal Mail Steam Packet Co.
Meduna	10,123	482	59	28	150	Cie de Nav. Ind. Atlan. Co.
Aorangi*	17,491	580	72	27'-10"	150	Union Steamship Co.
Omaha	.....	...	...	.....	193'-0"	U. S. Navy
Saratoga	.....	...	...	.....	185'-0"	U. S. Navy
Lexington	.....	...	...	.....	185'-0"	U. S. Navy
Medusa Auxiliary	.....	...	...	.....	170'-0"	U. S. Navy
Colorado	.....	...	...	.....	168'-6½"	U. S. Navy
California	.....	...	...	.....	168'-3"	U. S. Navy
West Virginia	.....	...	...	.....	168'-0"	U. S. Navy
Whitney Auxiliary	.....	...	...	.....	167'-0"	U. S. Navy
Bridge Auxiliary	.....	...	...	.....	166'-0"	U. S. Navy
Pensacola	.....	...	...	.....	164'-0"	U. S. Navy
Dobbin Auxiliary	.....	...	...	.....	163'-0"	U. S. Navy
Vestal Auxiliary	.....	...	...	.....	158'-0"	U. S. Navy

* Steamers which have called at Pacific Coast ports of U. S.

provided for vessels now entering the port but the future must be considered, since the channel may be deepened, allowing vessels of deeper draft to enter. Also, allowance must be made for possible rerouting of existing lines. For instance, the present gigantic Atlantic liners may be transferred to Pacific service either because of their being needed there by the necessities of traffic or because of a war emergency when they would be needed for transporting troops.

The accompanying table contains data as to drafts and mast heights for a representative group of the larger vessels, including those requiring the highest clearance. Data as to mast heights of sailing vessels have been difficult to obtain, but all sailing vessels are so constructed that their topmasts can be housed. Many heights of sailing vessels appear to range up to 171 ft., which was the maximum on which data could be secured.

Due to changes and alterations in rigging the mast heights of individual vessels as given in the table may vary slightly, but such a variation for a single vessel will have but little effect upon the conclusions reached as to bridge clearances by considering as a group the figures on mast heights shown in the table.

Acknowledgment for furnishing data as to mast heights is made to H. T. Cole, U. S. Merchant Fleet Corporation; Capt. L. B. McBride, U. S. Navy; and J. H. Polhemus, general manager and chief engineer, Port of Portland.

$200,000,000 for Industrial Research

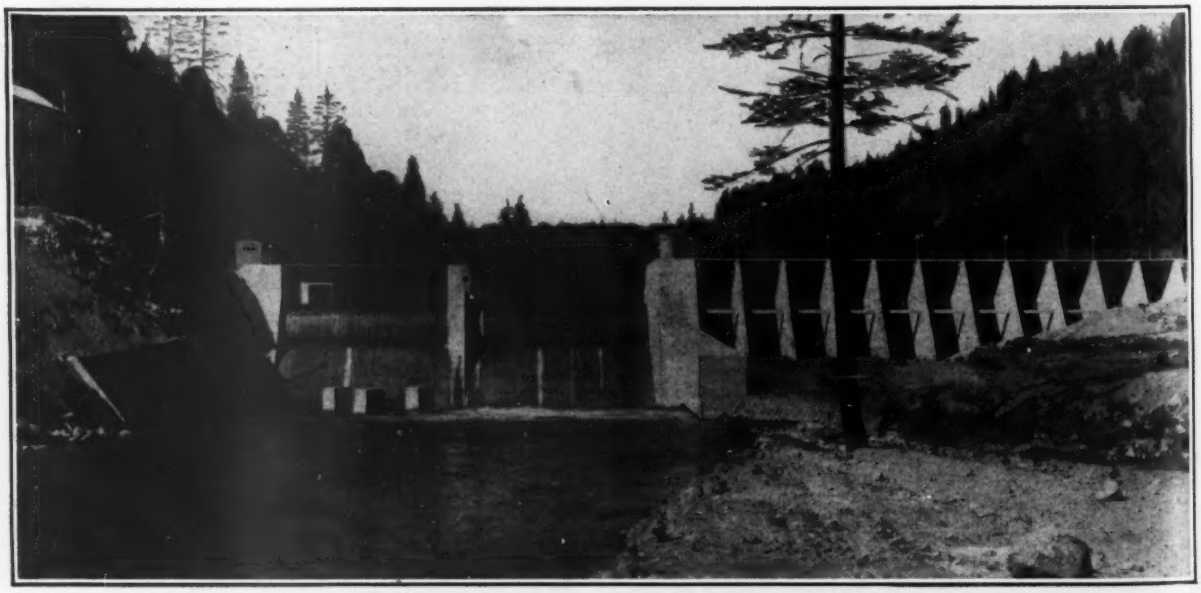
Industrial research by corporations and by the federal government represents an annual expenditure of $200,000,000, according to the National Industrial Conference Board. Of this amount industry spends two dollars for each one dollar spent by the government. Further, the report says that: "Whereas in 1921 only 578 companies were known to maintain research departments or laboratories, more than 1,000 companies have organized research divisions in operation at the present time. In addition, 70 trade associations are spending about $15,000,000 a year in research and 152 colleges and technical schools about $1,500,000. Much technical research work is also accounted for as part of the production costs of industrial organizations and is not included in the total sum here given. Industries whose research expenditures were largest five years ago are those which have scored the greatest relative growth since then. One manufacturing company in 1926 spent $5,250,000, while a public utility corporation spent approximately $13,000,000 within the same year. Manufacturing companies were found to spend on an average of from 1 to 3 per cent of the gross receipts from sales on research. Other large sums spent for research, not included in the above title, are credited to life insurance companies, privately endowed social, economic and charitable institutions, state and local governments, and commercial research organizations selling their services to others on a fee basis."

Composite Gravity and Hollow Type Dam on Pit River

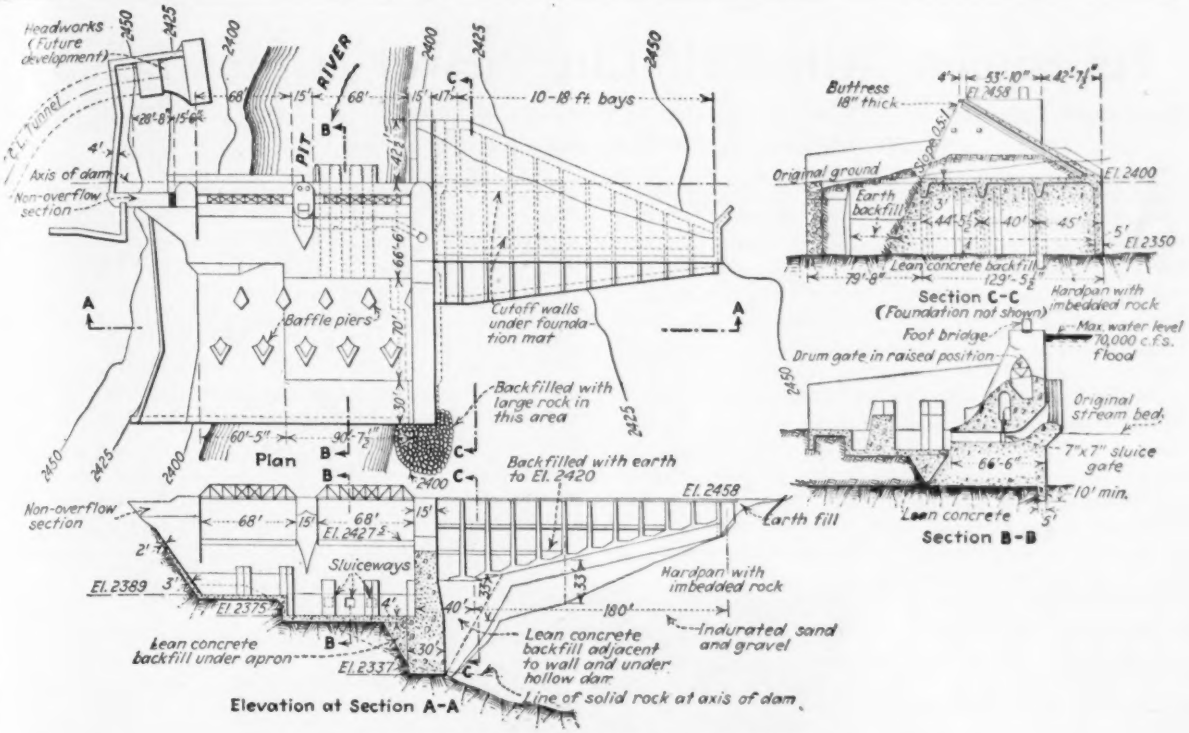
Foundation Conditions Necessitated Hybrid Structure—Design Combines Security and Economy

THE PIT 4 dam completed by the Pacific Gas & Electric Co. on Pit River, Calif., in July, 1927, was designed for a site at which there is solid rock foundation on one side of the stream and comparatively soft underlying strata on the opposite bank. The combination gravity- and hollow-type dam selected for this site kept the cost within reasonable limits at the same time that it fully satisfied foundation requirements.

For the present the principal function of the structure is to form a 2,000-acre-ft. afterbay for the Pit 3 power house. Ultimately it will be the forebay for the Pit 4 development which is to be constructed below. The dam had to be built at the present time in order that compensation might be made in the afterbay control for the fluctuating flow through the Pit 3 power house caused by daily load variations. The power company is



DOWNSTREAM SIDE OF DAM AFTER COMPLETION
Baffle piers in lower spillway bay are submerged.



DETAILS OF THE PIT 4 DAM IN CALIFORNIA

under contract for uniformity of flow because of the needs of irrigation districts below this development.

Bedrock appears at the ground surface on the right bank of the dam site but drops below the stream to a depth of about 50 ft. at the left edge of the river channel, increasing in depth toward the left bank. The gravity portion of the structure is founded on this bedrock and the spillway is included in this part of the dam. Between the gravity- and the hollow-type sections there is a solid concrete division wall, founded on rock, 30 ft. thick and 170 ft. long at its base and 120 ft. high at the highest point. This wall serves to some extent as an anchorage for the outer end of the gravity section of the dam and also as a protective and retaining wall for the river edge of the hollow-type structure.

The hollow-type portion includes a 3-ft. reinforced-concrete slab over a considerable area of earth base, protected by a concrete cutoff wall extending to hardpan with embedded rock. After the completion of this portion of the dam, backfill was placed with a dragline in the several bays to add more weight, this fill being brought up to El. 2,420.

Details of the dam are given in the accompanying drawing. One notable feature is the use of lean concrete for backfill under the reinforced-concrete slab of the hollow-type section and under the floor of the stilling pool below the spillway section.

The downstream toe of the overpour section is protected by a broad concrete apron, built at two levels, the better to conform to the slope of the bedrock on this bank. On the apron are located baffle piers to dissipate the energy of the overflowing stream. The design of these piers is similar to that developed for the Pit 3 dam (see *Engineering News-Record*, June 3, 1926, p. 886).

The excavation at the dam site totaled about 70,000 cu.yd. and the concrete yardage poured totaled 42,000, of which three-fourths went into the gravity portion of the structure, thus indicating the concrete saving due to the combination design adopted.

The dam was designed and constructed by the engineering organization of the Pacific Gas & Electric Co., A. H. Markwart, vice-president in charge of engineering, I. C. Steele in charge of design, and O. W. Peterson, construction engineer.

Status of Helium Production

Production of helium for the Army and Navy is carried out by the Bureau of Mines, of the U. S. Department of Commerce. This work began ten years ago, up to which time helium had been obtained only in small amounts as a curiosity in scientific laboratories. Costs of production at that time ranged from $1,700 to $2,000 per cubic foot, and practical use of the material was out of the question. Then, however, the process of recovering helium from natural gas, which in certain wells was found to contain small quantities of the rare gas, was developed on a commercially feasible scale, and a helium production plant was erected at Fort Worth, Tex. This plant went into operation late in 1922 and has been producing helium since. It was under jurisdiction of the Navy Department until 1925, and since then under that of the Bureau of Mines. During the year 1925-6 the plant produced 9,355,623 cu.ft. of helium at an operating cost of 3.41c. per cubic foot, but in the following year the production declined to 6,330,000 cu.ft. and the cost went up to 4.37c. Other helium sources are now under study, according to the annual report of the Bureau of Mines, just issued. It is expected that from these sources larger quantities of helium may be produced for the War and Navy Departments, at a very much lower cost per unit.

Seventeen-Mile Belt-Line Sewer at Springfield

Quicksand Tunneling and Deep Cuts in Caving Ground Chief Difficulties of Large Interceptor Which Includes Sewer on Trestle Over Swamp and in Siphon Under Creek

A VARIETY of materials and a number of special structures in the new intercepting sewer system at Springfield, Ill., make this work of exceptional interest. These sewers include monolithic concrete, concrete pipe, vitrified segmental tile-block and vitrified pipe. Their structures include a reinforced-concrete trestle aqueduct 2,100 ft. long; a tunnel 4,455 ft. long with vitrified tile-block lining; an inverted siphon with differential pipes; two pumping stations; and numerous intercepting devices of different types to separate the dry-weather and storm-water flows at connections with the city sewers. A sewage-treatment plant is under contract and a storm-water treatment plant is proposed. The variety of sewer materials used is due to the prices bid for alternative materials on the several contract sections. Figs. 1 and 2 show some of the construction work and Fig. 3 is a map of the system.

Springfield, with a population of 70,000, has eleven main sewer outlets discharging about 6 m.g.d. into Spring Creek and Sugar Creek, the latter flowing into the Sangamon River above the waterworks intake. To prevent this pollution of streams and water supply, two intercepting sewers are being built on the east and west sides, leading to a sewage treatment plant on the north, as shown in Fig. 3. As the city's sewers are on the combined system, these interceptors are designed to carry the first runoff of storm-water flows in addition to the dry-weather flow. On each interceptor there is a pumping station with a 20-ft. lift.

FIG. 1—TRENCH EXCAVATOR FEEDING TO CONVEYOR BACKFILLER

Intercepting Sewers—The east side interceptor, 36 to 66 in. in diameter, is 39,000 ft. long, with sewer connections aggregating 3,800 ft. It is designed for 25 times the dry-weather flow (or 2,125 g.p.d. per capita) for the stretch leading to the Cook St. pumping station. A future storm-water treatment plant is proposed at this station, so that the treated overflow can be bypassed into the creek (Fig. 3) and only the ultimate dry-weather flow of 300 gal. per capita pumped over the divide to the sewage-works. The west side interceptor, 15 to 69 in. in diameter, 51,100 ft. long, with 11,300 ft. of connections, is designed for an ultimate capacity of 425 g.p.d. per capita. Typical sections are shown in Fig. 4. The bids covered alternative materials as follows: Monolithic concrete, brick, reinforced-concrete pipe and vitrified segmental tile block for diameters of 69 in. to 36 in.; reinforced-concrete pipe and vitrified segmental tile block for diameters of 36 in. to 27 in., and vitrified pipe for diameters of 24 in. and less.

Open-Cut Construction—Trenching on division A, with a cut of 6 to 20 ft., was done at first with a ¾-yd. dragline machine; a similar ½-yd. machine did the backfilling. But for the heavier work a trenching machine was used in combination with a special backfill conveyor. This conveyor (Fig. 1) consisted of a steel frame about 100 ft. long mounted on wheels and carrying a belt driven by a small gasoline engine. The belt received spoil from the lateral belt of the excavator and delivered it to a chute leading into the trench. This is said to be the longest conveyor ever used in backfilling. In fair weather, the ditcher could cut about 300 ft. of trench 5 ft. wide and 16 ft. deep in a 10-hr. day. In wet weather progress was retarded by the conveyor, as the belt would slip and the shale or clay would stick to the belt.

With the excavator, conveyor and 36 men working on a trench 6 to 14 ft. deep, the average daily progress over a long period was 80 ft. of 36-in. pipe laid on a 4-in. bed of concrete. The highest daily record was 338 ft. with a 6-ft. cut. On the second half of the work, where the cut averaged 14 ft., there were continual difficulties. A large amount of ground water was encountered and the banks caved so that the top of the trench was 50 to 60 ft. wide. To meet these conditions the contractor drove lines of interlocking steel sheeting in advance of the trenching machine and excavated between them. With this plan, the completed sewer averaged 20 ft. per day.

Pipe laying kept close to the machine, the pipe being handled by a crawler crane and laid directly on the ground in good material or on a 4-in. bed of wet concrete in soft ground. A small mixer moving along the side of the trench chuted the concrete into place. As the machine advanced the sheeting was pulled by the crawler crane about 50 ft. behind it, and the earth at once caved in. Backfilling was completed by the conveyor.

On other contracts, dragline excavators were generally used, but there was considerable delay due to the amount of rock and the necessity of sheeting clay

FIG. 2—ELEVEN-FOOT SEWER ON TRESTLE OVER SWAMP

trenches more than 10 or 12 ft. deep. In one case, a 69-in. monolithic concrete sewer was in a 28-ft. cut with 26 ft. of solid rock, which was blasted and removed by the dragline machine.

Steel forms in 50-ft. lengths and portable mixing plants chuting the concrete into place were used for the monolithic sewers. For 66-in. diameter the daily progress ranged from 16 to 60 ft., except that on curves it did not exceed 30 ft. With segmental tile block, progress was 27 to 75 ft., depending largely upon the excavator. With two masons and two helpers in the trench and seven men on top, an average of 60 ft. of 57-in. sewer could be laid daily, when not hampered by the trenching.

Subdrainage in Wet Ground—Where the soil was very soft and muddy a 4-in. tile drain was laid under the sewer, requiring special construction, as shown in Fig. 5. The trench was excavated by a dragline machine to about 2 in. below the bottom of the sewer barrel, a 12 x 12-in. trench being dug by hand for a 4-in. tile drain surrounded with gravel. The ground was so soft that the sides of this small trench would flow when men walked along the main trench. After the drain was laid, stepping on a tile would sink it entirely below grade, and stepping beside it caused the tile to be pushed out of line.

To keep the drain in grade and line, it was laid in a V-trough made of two planks 1 x 8 in. and 1 x 6 in. nailed together, as shown by Fig. 5, the trough and drain being then surrounded with gravel. In the worst cases, a 21-in. cover of 1-in. planks cleated together was placed over the subdrain. This served to prevent the gravel from being clogged with mud and concrete; it also provided a more stable foundation for the sewer and a working platform for the men in laying the bottom half of the sewer. This cover was used only under segmental-tile sewers, but the V-trough was used with various types of construction. The subdrain carried the seepage to a sump in the side of the main trench. Whenever the drain became clogged a new sump was made at the upstream end of the obstruction and another line of drain was started. This method of construction for extremely soft ground was devised by L. A. Wilson, resident engineer, and proved very satisfactory.

Slides in Trench—At one point a trench 26 ft. deep in sidehill ground was distorted by a slide, so that its top was 100 to 150 ft. wide and the completed 66-in. monolithic concrete sewer was raised about 5 ft. above its grade. To stop the slide, wood sheeting 9 in. thick was driven on the uphill side to a depth of about 28 ft. or into the solid ground below the slide. This sheeting was braced against the opposite side of the trench. A length of 100 ft. of the sewer was then broken up and rebuilt at grade with walls 24 in. thick. The inside forms were heavily braced horizontally and vertically, the outside forms were left in place and backfilling was done the same day or 12 hours after pouring the concrete.

At another point, on division A, the heavy spring

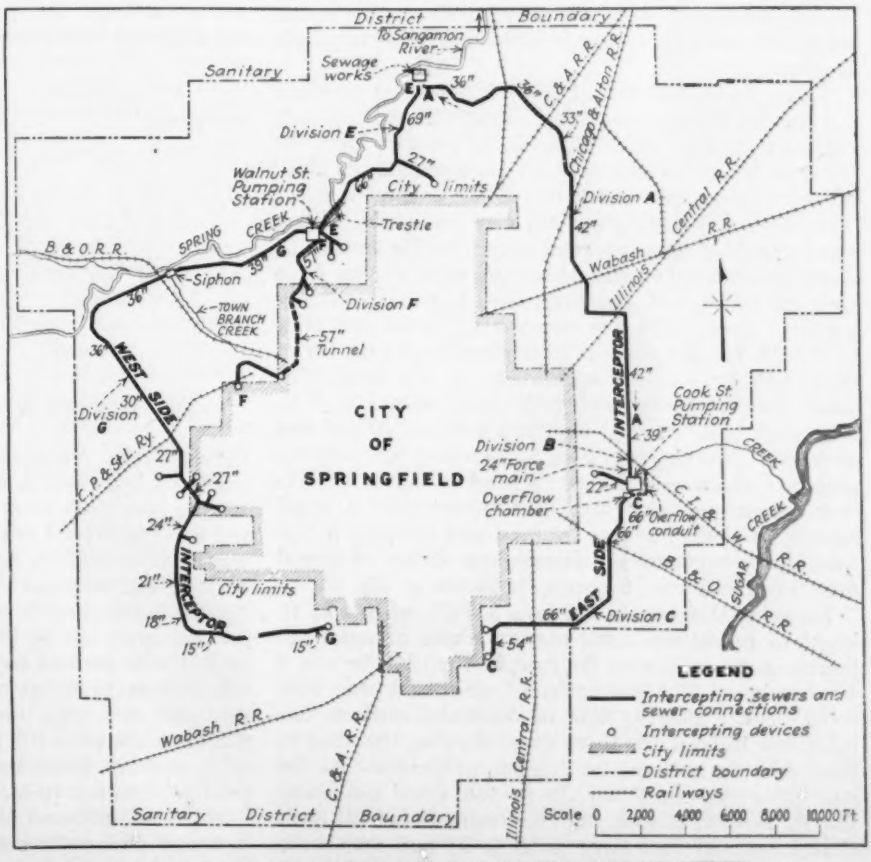


FIG. 3—LOCATION MAP OF BELT LINE INTERCEPTING SEWER

rains caused a slide on a trench 23 ft. deep that was being cut by a dragline machine as an approach to a short tunnel under the Wabash Railroad. This slide wrecked the timbering of a shaft for the tunnel and endangered the railroad so that a pile trestle had to be built to carry the tracks while the sewer was built in an open cut, with steel sheeting. To prevent similar trouble on the opposite side of the railroad, two lines of steel sheeting were driven for a distance of nearly two miles and the trench was excavated between them.

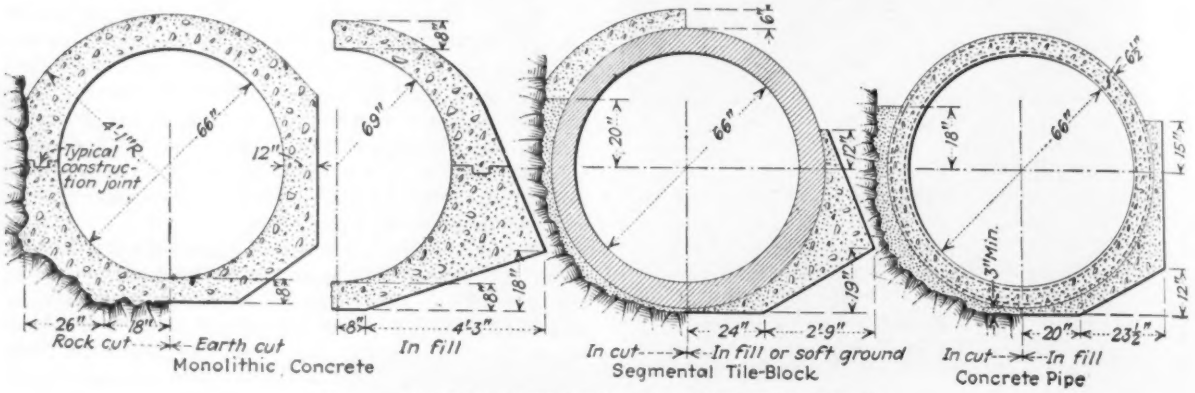


FIG. 4—TYPICAL SECTIONS OF INTERCEPTING SEWERS

Sewer on Fill—On division E, about 650 ft. of 66-in. monolithic concrete sewer is in a fill with a maximum height of 15 ft. This fill was built with earth excavated from borrow pits by a steam shovel and deposited by wagons with teams, which served to compact the material. As the fill was built about three months in advance, the sewer trench cut in it stood up with very little bracing.

Sewer Tunnel—A tunnel 3,500 ft. long was provided for division F, but owing to difficulties with the approach cut the contractor drove 4,455 ft. in tunnel. Here the sewer is 57 in. in diameter, built of segmental tile block backed with concrete which fills all the excavated space. This work was mainly in hard clay and shale, with a short stretch of limestone, and a considerable amount of water was encountered. Headings were driven from each end and in both directions from two shafts averaging 50 ft. deep. The clay was covered by the unit price of $28.75, but the shale and limestone were paid for at $6 and $7 per cu.yd. respectively. In the dense blue shale, compressed-air hand drills made holes 5 to 6 ft. deep, each being charged with two sticks of 40 per cent dynamite. Mucking was done by shoveling into narrow-gage cars which were pushed by hand to the shaft. Except where quicksand was encountered only a small amount of timbering was required and although a few small slides occurred no damage was done. A portal with three-piece roof timbering is shown in Fig. 6.

Tunnel in Quicksand—On June 3, 1927, when a 75-ft. length of tunnel was being lined, a trickle of water appeared in the roof, near the face, and quickly became a 1-in. stream. The indication of quicksand was then noted. After a losing fight of 36 hours, work at this point was stopped for three days, allowing the sand to drain and lose some of its dynamic properties. As the sand filled the tunnel for 30 ft. and tailed out about 100 ft. further, it took the crew four 10-hr. shifts to clean it out and then three shifts to fill and timber the space above the tunnel, which was about 20 ft. wide and 8 ft. high. The sand vein was about 4 ft. thick, with its bottom 4 ft. above the roof and a dip of only about 4 deg. in the direction of the heading. Stringers of 10-in. round timbers were placed at the bottom of the sand vein, and the space above them filled with cull tile and baled hay, and thoroughly tamped. The heading was then timbered with three-piece sets of 3-in. oak, braced vertically and horizontally at the ends of the caps. The space outside was then filled in the same way as the roof cavity. Tunneling was resumed without blasting, working three 8-hr. shifts.

Soon after, the sand became so fine as to seal off the water entirely, and hydrostatic pressure increased until it was difficult to hold back the sand. When the drift had advanced about 20 ft. a second cave occurred. No effort was made to fill the space, as it was evident that there would be one continuous cave until the tunnel had passed

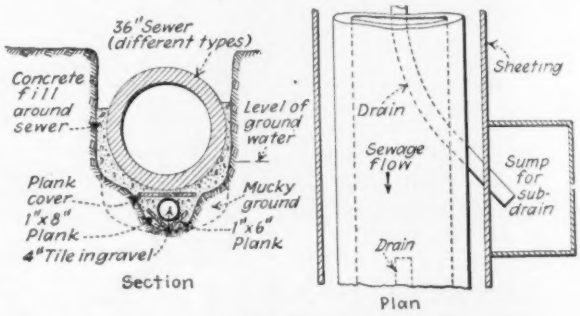


FIG. 5—TRENCH SUBDRAINAGE IN SOFT GROUND

the sand vein. Timbering was continued with the three-piece sets and about 2-ft. of hay and cull tile over them.

Four such caves occurred in a length of 190 ft. When the vein had dipped to within 1 ft. of the roof no effort was made to hold the sand. Timbering was within 6 or 8 ft. of the face, and the sand and shale were mucked out, the latter being thrown over the top of the timbering into the space left by the sand. This cavity was finally packed with tile and hay. The sand vein ended before it had dipped to the springing line of the sewer. Through this quicksand, with 6-man crews, it took 147 shifts of 8 hours to complete 105 ft. of sewer. At all places where quicksand was encountered the segmental tile block was reinforced with a 12-in. ring of 1:2:4 concrete.

Near the south end, the approach cut was 20 ft. deep, in a soft sticky clay that flowed when wet, and it took four days to build 30 ft. of sewer with a dragline and 14

men. It was thought that the job could be handled by sheeting the trench and using a clamshell bucket, but the contractor preferred to tunnel for a distance of 400 ft. He made about 7 ft. of sewer per 10-hour shift, with little difficulty, although no timbering was used. The excavation was done by hand, as the clay was too sticky for pneumatic tools.

Concrete Trestle Aqueduct—A structure of exceptional interest is a closed reinforced-concrete aqueduct trestle, 2,100 ft. long, carrying the 66-in. west-side interceptor over Spring Creek at a height of about 8 ft. This is shown in Figs. 2 and 7. As this valley is to be included in the park and boulevard system, attention was given to effective design of the piers and spans, the latter being made of octagonal section on this account. Special problems were involved in the design of the 66-in. barrel as a beam supported on piers 45 ft. c. to c. and

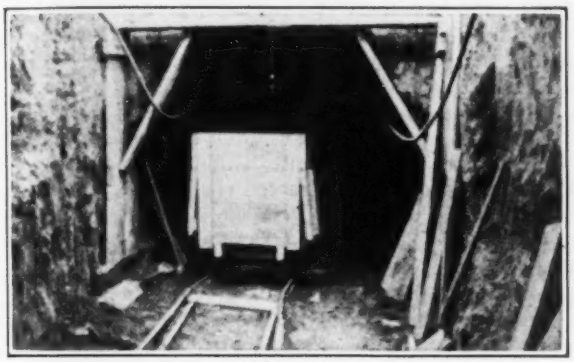


FIG. 6—THREE-PIECE TIMBERING IN TUNNEL

with an expansion joint over each pier. It is heavily reinforced as a beam and provided with hoops to resist any tendency to flatten. Load tests showed satisfactory deflection. Piles are used in soft ground, but the entire length is over old coal-mine workings where subsidence may occur.

This trestle consists of 46 spans of 45 ft. each, with an inside diameter of 5½ ft. and the outside octagonal with a least diameter of 7 ft. Each pier consists of two posts on a single footing and supporting an arched beam or cap, while extensions of the posts are carried up the sides of the barrel. The piers average 11 ft. in height, but are 21 ft. high at the two creek crossings, where the footings are 4 ft. below the bed. Piles were used under the four creek piers and nine piers on soft ground, the others being proportioned for low bearing values of the footings. Expansion joints in the barrel are built up of three ½-in. layers of bituminous felt around a copper diaphragm with bituminous filling.

In tests under dead-load only the maximum deflection at center of span was 0.133 in. and the total flattening at top was 0.016 in. With a 7,000-lb. gasoline locomotive and 4,000-lb. car of concrete on a span, the additional deflection at one center was 0.005 inch.

Concrete Trestle Construction—Half of the footings were built with a portable mixer and stock piles on the ground, but as the site is subject to overflow the main mixing plant was located upon a hill at one end, the concrete being handled by an industrial railway. Pier excavations averaged 4½ ft. and were so wet that they had to be drained to sumps for continuous pumping while the concrete footings were poured. At the creek crossings the depth was 14 ft. and cofferdams of wood sheeting were built, the excavations being then done by a crane with ¾-yd. clamshell bucket.

Exceptionally substantial falsework was built for the spans, to guard against vibration in the green concrete due to the movement of the industrial cars. Each span had five two-pile bents of 25-ft. cypress piles driven to 20-ft. penetration, the bents being 7 ft. apart. Ten lines of 2x10-in. joists carried by the bents and the pier copings supported a floor of 1-in. plank on which the sewer barrel was built. During the construction of the piers, a lighter floor system served to carry the industrial track. The two end bents of falsework were removed after 14 days, but the three middle bents were left in place 28 days.

Two sets of wooden removable forms were used for the 45 piers. In cold weather, only one pier was built each day, being then enclosed within a portable wooden housing and kept warm with salamanders for 24 hours, while the forms were left for 48 hours. An oil-heater discharged hot air into the mixer and concrete was kept in the drum for three minutes to insure thorough heating. With warm weather, two piers were built in a day and the forms were removed after 24 hours. Concrete was discharged into cars holding six batches, the cars being pushed by hand to the forms, as the temporary floor was not strong enough to carry the locomotive. All but four of the piers were built during the winter and early spring, so that when fair weather came the barrel construction could proceed without interruption.

Building Aqueduct Spans—Three sets of removable outside forms and one set of collapsible steel inside forms sufficied for all the spans. The former were of tongue-and-grooved plank, well oiled on the inner surface. The latter were in 30-ft. lengths. With the outside forms erected, the steel reinforcement was placed, consisting

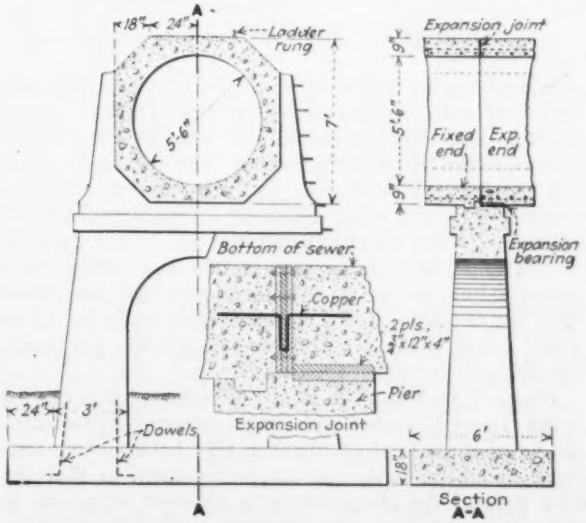


FIG. 7—SEWER STRUCTURE ON TRESTLE, SHOWING EXPANSION JOINT AT PIERS

of 14 longitudinal bars in the bottom, 54 hoops and 108 transverse bars in top and bottom, all tied to longitudinal temperature bars 18 in. c. to c. Then the lower quarter of the barrel was poured, using a dry mix that could be shaped to the invert by a screed, the surface being float-finished by hand. The screed slid on 4x4-in. guide rails on brackets attached to the forms and serving to make a groove in the construction joints.

After this concrete had set for 24 hours, the inside forms for the upper part of the barrel were carried forward by a traveler on a track and were placed directly on the concrete. They were bolted to the outside form to prevent bulging, but the bolts were pulled after 24 hours and the holes then filled with mortar. The inside forms were left in place 24 hours before being moved ahead. The two lines of outside forms were held together at the top by 6x6-in. timbers on which the industrial track was laid. The concrete cars for this part of the work were handled by a locomotive. After three days these forms were removed, the track being then placed on the completed barrel. During cool weather the expansion joint filler served as a bulkhead at the ends of the forms, but in hot weather the bituminous material became so soft that elaborate bracing was required to hold it in position.

Gravel concrete of 1:2:4 mix was used. Its water-cement ratio was kept as low as possible, 5½ gal. per sack of cement for concrete below the construction joints and 7 gal. for concrete above these joints. The concrete

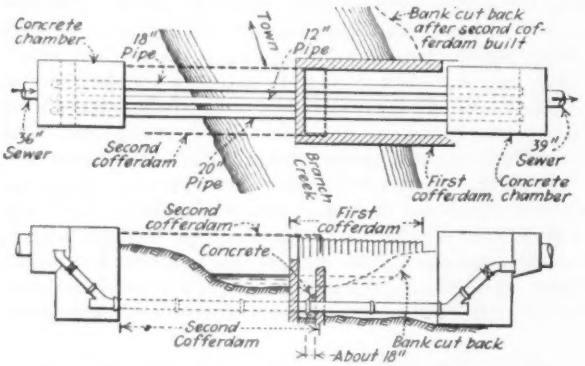


FIG. 8—SIPHON CONSTRUCTION BY COFFERDAMS

was well spaded and tamped around the steel, but very little free water flushed to the surface. To prevent segregation, the concrete chuted into the batch boxes was stirred in the boxes with shovels. For the upper work, the construction joint was carefully cleaned and then covered with rich cement mortar before depositing the concrete. After three days the outside forms were removed and the concrete was wetted and rubbed with carborundum brick; it was then sprinkled five times daily for two weeks. No leakage developed in the filling tests. At first the progress was four spans per week, but this was increased to six spans.

Inverted Siphon—The west-side interceptor is carried under Town Branch Creek by an inverted siphon consisting of three lines of cast-iron pipe between two concrete chambers. An unusual arrangement is that the inlet ends of the pipes are set at different elevations in order to maintain proper velocities with different rates of flow. The lowest or 12-in. pipe will suffice for flows up to a velocity of 3.5 ft. per second; by that time the sewage will have risen to the 18-in. pipe, which will carry 4.4 sec.-ft. Additional flow will pass through the 20-in. pipe with a maximum velocity of 4.3 ft. per second. A loss of head of 0.7 ft. is estimated with the siphon operating at 13 m.g.d., its full capacity.

Difficult Work on Siphon—In laying the three parallel lines of cast-iron pipe, the main problem was that of handling the creek, which is fed by a 16-ft. sewer carrying more than half the city's sanitary and storm-water flow and which rises rapidly after a small shower. A dam was first built about 50 ft. upstream from the siphon, raising the water level about 4 ft. and diverting the flow through a draw or channel which led into the stream about ¼ mile below. But this dam was demolished by the flow after the first heavy rain, which occurred soon after this work was finished. The next plan was to build a sluiceway 12 ft. wide at stream bed elevation and drive sheetpiling from the ends of the sluiceway to each bank. It was intended to pump out the enclosed space on each side of the sluiceway, and then lay the 60-ft. length of pipes in these spaces and under the sluiceway. But again a heavy rain came and 80 per cent of this work was washed out.

The third and successful method, shown in Fig. 8, was to build a cofferdam in half the width of the stream, using interlocking wood sheeting composed of two 1x8-in. planks with a 2x8-in. plank between them. After two 12-ft. lengths of pipe were placed, the lower ends of the piling were concreted and a second end wall was built about 2 ft. back from the ends of the pipes. The piling of the side walls was then pulled and redriven to form a cofferdam in the other side of the channel, with the pipes projecting into it. Three lengths of pipe in this second cofferdam served to reach the siphon chamber.

Interceptor Connections—To divert the dry-weather flow into the intercepting sewers, intercepting devices are placed at the various connections with the existing city sewers. These devices include the overflow weir, orifice and leaping weir types. In general, the first is used where head is limited. The others are used where considerable head could be obtained easily, with relatively short extensions of the connections with existing sewers, and where comparatively close control of the amount of intercepted flow was necessary. The upper end of division F connects with a 16-ft. horseshoe sewer having a maximum capacity of 3,000 sec.-ft., but where the maximum flow to be intercepted is only 50 sec.-ft. At this point it was necessary to use a chamber fitted with float-control regulators of the orifice type, designed to close the connection entirely when the flow in the large sewer reaches a depth of 8 feet.

Pumping Stations—Each of the two sewage pumping stations contains four vertical-shaft centrifugal pumps with direct-connected motors. Those at the Walnut St. station, pumping directly into the concrete aqueduct, have capacities of 2,000 to 8,000 g.p.m., aggregating 19,000 g.p.m. At Cook St. the capacities are 1,200 to 5,500, aggregating 12,500 g.p.m. The lift is approximately 20 ft. in each case. The pumps are mounted in a dry well or chamber having a suction connection to a wet-well, with automatic start-and-stop control of the pumps by float connections to the switchboard. In addition there is a similar small unit of 150 g.p.m. with a sump for draining the dry well. Both stations are fully equipped with meters for power and sewage flow.

Contractors and Engineers—To carry out the project for intercepting and treating the sewage, the Springfield Sanitary District was organized in 1924. A bond issue of $1,500,000 was approved and an additional $800,000 will be obtained from general taxation. The works were planned by Pearse, Greeley & Hansen, Chicago, as consulting engineers for the district. For the five divisions of intercepting sewers the contractors were: Division A, Hanson & Willadsen, Omaha, Neb.; B, C and G and Cook St. pumping station, Sangamo Construction Co.,

Springfield, Ill.; div. E, Floyd Shofner, Atchison, Kan.; div. F, H. C. Willadsen, Omaha, Neb.; Walnut St. pumping station, Merkle Contracting Co., Kansas City, Mo.

W. B. Walraven is district engineer for the Springfield Sanitary District, with W. E. Dawson, F. P. Schrader, C. C. Wood, Ray V. Tilly and L. A. Wilson as resident engineers.

Thirteen Points to Be Covered in Land Drainage Report

Expansion of Outline First Drawn for Guidance of State Engineers of Louisiana

By Arthur M. Shaw
Consulting Engineer, New Orleans, La.

As a possible aid to others, the following outline of reports for land reclamation by drainage is submitted. This outline is one which the writer has used for a number of years in the preparation of reports on proposed developments under various state drainage district laws. It is based on (though somewhat expanded from) an outline originally prepared by the late Frank M. Kerr when president of the Board of State Engineers of Louisiana for the guidance of engineers in the preparation of reports to be passed on by his board: (1) Location and boundaries. (2) Estimated area. (3) Classification of lands, (a) as to topographic conditions, timber, etc., (b) as to ownership. (4) Fertility of soil. (5) Present drainage facilities and operation. (6) Feasibility of reclamation. (7) Description of recommended works. (8) Estimated cost of items and total. (9) Estimated cost per acre. (10) Cost to put land in shape for farming. (11) Cost to maintain drainage. (12) Estimated value of land after reclamation. (13) Conclusion and recommendations.

While the outline is followed in practically all projects handled by this office, the amount and degree of detail covered under each head is controlled by special conditions and requirements. For ordinary drainage district projects it usually is desirable that the initial report be sufficiently in detail so that (with its accompanying drawings) it may serve as a basis for spreading assessments for benefits, letting contracts and all other work leading up to actual construction. Frequently, lack of available funds makes it impossible to carry the studies to this point at the outset, but under most conditions it is safe to assume that if the project does not appear to have sufficient merit to justify the owners in incurring the expense of a thorough examination, it should be left for future development.

Details of treatment under some of the headings, as practiced in this office, are given in the following:

(1) *Location*—A map accompanies the report which shows the boundaries of the proposed district with as much topographic detail as may be of interest to owners, officials and prospective bidders. In one corner of this map, or as a separate sheet, a key map is given, showing nearest towns, location of railways and approaching highways and other data which may have a bearing on the delivery of construction equipment and supplies. In addition to the maps, a description of the boundaries of the district should be given in the body of the report, this description being of such a nature that the boundaries could be established on the ground without reference to the map.

(2) *Area*—If the district boundaries follow regular section lines, the area is computed from United States Land maps, but if the boundaries are irregular, and a more suitable system is not practicable, planimeter measurements from a map drawn to a reasonable scale are used, explaining the conditions in the report in order that it may be understood that the area as stated is only reasonably approximate. I never have considered it as a part of the duties of an engineer employed to plan a drainage system to run out, in detail, the various property lines. Under some conditions, this easily might cost more than all of the other field work.

(3) *Classification*—Data of this nature should be shown in tabulated form, with key references to the map to designate the various subdivisions. Even where necessary to make rather extensive explanations of column headings in the text, the tabulation of data which lend themselves to that form is used.

(6) *Feasibility of Reclamation*—Considering that this covers the economic justification, as well as the physical practicability of the proposed works, it forms the crux of the entire matter. While it can be covered in one or two sentences, these should be definite and conclusive.

(7) *Description of Works*—Unless the preliminary surveys have been made quite in detail, it usually is better not to fix the location, dimensions and description of proposed works with more definiteness than is necessary to indicate in a general way their character and general location, as the construction forces should have some latitude in making minor adjustments.

(8) *Estimated Cost*—While this is not given in minute detail, a reasonable analysis is submitted. An effort is made to adjust each item of the estimate to such a figure that it may not be necessary to include an unreasonably large item to cover unforeseen contingencies. The usual practice is to include one item (amounting to from 10 to 20 per cent) to cover legal and engineering expenses and contingencies. With a proper treatment under the heading, "Description of Work," very little should be required under "Estimated Cost" which cannot be included in the tabulated estimate.

(10) *Cost to Put Land in Shape for Forming*—Under most drainage laws, it is contemplated that the community effort represented by the work done by the district shall include only works of a character which requires special talent and equipment and which cannot be done economically nor efficiently by the individual land owners. This includes dredged canals, major tile outlets, pumping plants, levees, etc., but ordinarily does not include clearing of timber and the construction of field ditches, branch lines of subdrainage and similar works. A report should take cognizance, however, of such works as may be necessary for the proper utilization of the land.

(11) *Cost to Maintain Drainage*—This probably is the most difficult part of the report to prepare if any degree of accuracy is to be secured, but it covers a subject which should be considered by the promoters and a reasonably reliable estimate should be provided by the engineer. It forms an especially important feature if the plans include pumping plants for artificial drainage.

U. S. Army Air Corps

New Orleans, Looking Toward the Mississippi. Except for the Warehoses on the Levee, this whole Section is Below the River

FOURTH OF A SERIES OF ARTICLES:

Today in the Mississippi Flood Area

Engineering and Human Conditions in the Basins and Along the Levees as an Engineer Sees Them

By W. W. DeBerard
Associate Editor, Engineering News-Record

IV—What New Orleans Thinks of Flood Protection

WITH few exceptions, the people of New Orleans want spillways. They want them large enough to care for all water above a levee-full river surely —and just as much larger as can be obtained. They want also a diversion through the Atchafalaya. Many of them, and mostly the engineers, will go so far as to favor diversion of the whole flow down the Atchafalaya. Let New Orleans be an arm of the Gulf! Let salt water come up to Baton Rouge!

No one in New Orleans has much to say for reservoirs or for reforestation and so far as higher levees are concerned that is considered simply out of the question, as adding certain future disaster to present menace.

Quite frankly, Col. Marcel Garsaud, head of the Dock Board (as the Board of Port Commissioners of New Orleans is called locally), and his chief engineer, S. M. Young, state that it will cost $40,000,000 to raise the docks as well as cause a lot of other troubles. So higher grade for the levees is one of the things not desired. Dr. Isaac H. Cline, the veteran Weather Bureau forecaster of New Orleans, has prepared a paper for the American Meteorological Society in which he will show diagrams to indicate that higher levees have coincided with higher flood stages.

Reforestation was not even given consideration in my hearing, during a week in the city spent in interviews with all kinds of people from bank presidents to cane sugar experts and, of course, including plenty of engineers. Perhaps the most satisfactory thing about the New Orleans situation is that laymen are willing to concede that the engineer can and will solve the flood problem. Laymen think he can do it quickly, while engineers are pretty much of the opinion that the problem needs many more data than are now available, particularly for many of the refinements.

The Back Levees

New Orleans needs better protection levees. Those who really know the situation as do the engineers were never much worried about the main river levees in front of the city. Col. Garsaud in his admirable description of the city's problem at the Columbus meeting of the American Society of Civil Engineers (see *Proceedings* for December, p. 2,577) dismisses the problem of levees above the city and back of it in a brief paragraph; nevertheless it is a plain fact that New Orleans has not the protection now at these points that it should have.

What might happen in case of a break of the main

river levees above the city, adding 2 to 4 ft. of water to Lake Pontchartrain perhaps on top of a Gulf storm tide of 5 or 6 ft., is a serious question. Steel sheet-piles by the hundreds were driven to add protection to the back levee system during 1927. Coming into the city from the north one passes a gap in such a line of sheet-piles.

George G. Earl and John L. Porter, respectively general superintendent and chief chemist of the Sewerage and Water Board, know how serious was the menace at that time. Mr. Porter stopped building additions to the filter plant for two months and turned over the whole force of the board to the levee work. Trucks hauled 20,000 cu.yd. of material out of a reservoir site for sand bags.

If better back-door protection is not provided for New Orleans before another flood it will not be because the engineers do not want it and have not preached about its need.

Governor Parker's Views

New Orleanans are keen for local protection but they are no less solicitious for a solution of the problem for the whole valley, said ex-Governor John M. Parker, whom I was fortunate enough to find in town—the one day in a month that I could have met him. When he is home from his strenuous speaking engagements (he has already given more than 75 addresses on the subject), "Mr. John," as every one affectionately calls the man whom the state authorities made dictator during the flood period, spends his time on his 4,200-acre plantation at St. Francisville. This is the location of several typical old-time Southern plantations. I spent a night in the town at one of the plantation mansions in the process of being converted into a hotel, and in the morning went to see "Rosedown," with its long avenue of live oaks lined with marble statues back of which was once a garden of magnificent grandeur, now largely grown up to underbrush. More than 500 varieties of roses were grown there; hence the name. This plantation is typical of those of many another of the ex-governor's neighbors, who have acres and acres of land but no help to keep the plantation going. A portion of the governor's place was inundated, so he knows what land owners experienced from personal contact. During the flood he spent shifts of 36 hours each at his desk with telephone girls in flooded towns moving up to the second floors for his eyes and the whole official force of the State of Louisiana at his command. John M. Parker knows the human side of the story as it was given few men to know it. He knows the problem of labor, of marketing, of fence wire that snaps in the hands from continued exposure to the flood waters and of the difficulties of coping with shoulder-high cockle burrs, the only weed that seems to have withstood the flood. Governor Parker naturally had little time but loaned me his only copy of the U. S. Chamber of Commerce referendum in which he said his views were expressed. After reading it carefully one is inclined to feel that the governor is responsible for much of the report. At any rate everything in it on the affirmative side coincides with what he told me.

Back Levee Protection—Going back to the back-levee problem and Mr. Earl's views is essential since this problem has not been stressed before. What worries him is the fact that hurricanes already experienced have raised Lake Pontchartrain levels 6 ft. above his drainage pumping station floors, also that for considerable lengths back levees begin to overflow at 2 ft. above station floor levels. Should a rain storm of the intensity of that of April 15-16, 1927, when 14 in. fell in 19 hours (of which 7.88 in. fell in 5 hours) that in itself is more than enough to cope with. In fact Mr. Earl for 10 years has repeatedly warned his board that 7 in. in 5 hours, which has now happened three times in 32 years (the period during which records have been kept) would cause serious flooding. It did. The drainage system can handle at full capacity intensities of 2 in. in one hour, 3½ in. in five hours or 7 in. in 24 hours. To care for an average rate of 1 in. per hour, as experienced on April 15-16, it is necessary to provide pumping and an outfall capacity of 30,000 sec.-ft., whereas the present capacity is 8,000 sec.-ft. As Mr. Earl states, providing such reserve capacity is both physically and financially impracticable. He suggested it as necessary only for the consideration of those who want 100 per cent results.

These facts are set down here, for they indicate what New Orleans has to cope with aside from any flooding due to breaks or overtopping of the 50 miles of back levees from storm heights in Lake Pontchartrain plus Mississippi water from crevasses above town. Were

FLOODING DUE TO HEAVY RAINFALL

flood heights up today and down tomorrow the chance of storms coming at the same time would be slim and might be taken, but the river rise is measured by 1 or 2 in. per day and falls at the same rate persisting for weeks.

On top of the hazard or chance of a back-levee flood accidents to equipment happen. One occurred on April 15-16 just as a rainfall of 7 in. in four hours started on top of 7 in. in the fourteen hours previously fallen. A power line of the local public service company dropped on top of the Sewerage and Water Board power line putting two 6,000-kw. generators out of commission and reducing the available power by one-half. For 30 days, until the coils of the burned-out generator could be renewed, the city was dependent on half power and one single unit for all sewage and drainage pumping. At thirty points wires of the company cross those of the board and Mr. Earl feels there should be municipal regulation to safeguard any more company lines falling on those of the board.

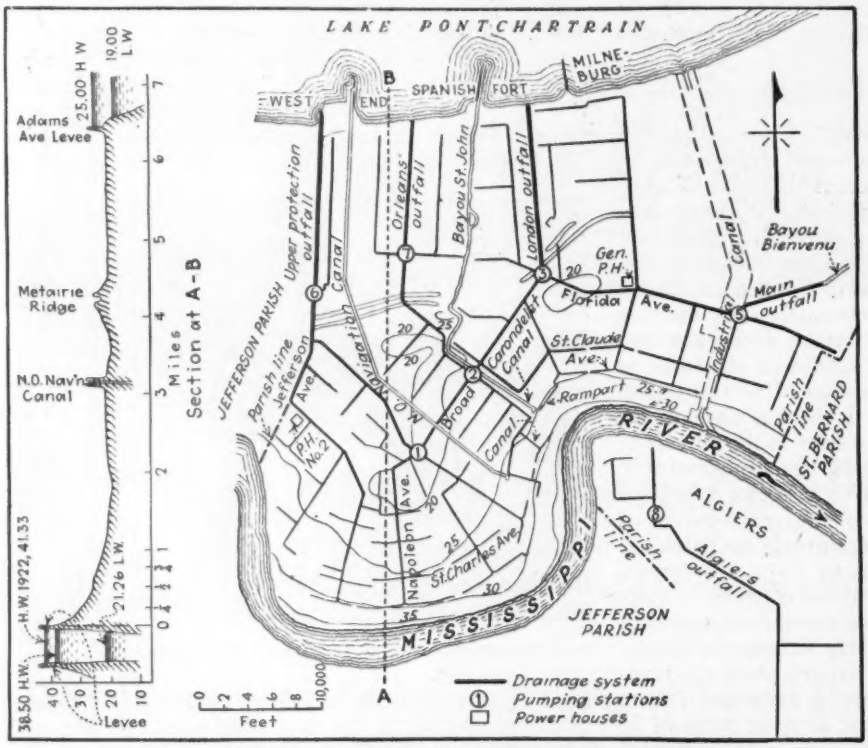
To protect the water purification plant and pumping station, steps were taken during the 1927 flood to permit them to remain in operation even though the surrounding ground should be inundated by 4 to 6 ft. of water. This precaution was taken because of the condition of the upper- and back-protection levees and the general fear

that the levees above the city would not hold if predicted flood heights were realized.

Another factor that is troubling the Sewerage and Water Board is the tendency toward scattered building without water, sewerage or drainage service over an area twice as large as is now served. While many other cities show the same tendency New Orleans with its intensity of rainfall, difficulty of drainage, and particularly potential menace of inundation, might well show the way toward reasonable regulation against dangerous decentralization. More than 7,500 scattered homes have no sewerage service while more than 10,000 available vacant sites have such service.

Spillway Advocates Numerous—"Ocular demonstration has convinced most of those not committed to spillways that relief of this kind is essential," said John Klorer, former city engineer, now commissioner of public works. He referred, of course, to the premeditated cut at Caermarvon, which is variously credited with having produced little or no effect to having prevented a crest of 2 or 3 ft. higher than that experienced. Mr. Klorer believes it kept water off the streets of New Orleans. People needed reassurance. Many had laid in provisions and moved cooking equipment to second floors. Banks having safety deposit boxes within easy reach near the floor level were besieged for space near the ceiling. Hundreds of trucks and flat cars were loaded ready to be rushed to points of danger. An extremely tense feeling existed which was dispelled by the cut at Caernarvon. It was worth $10,000,000 to the city in restoring confidence, in the opinion of more than one person interviewed.

As to spillways, John F. Coleman, veteran adherent of "levees only," still sticks to that theory. He very simply reiterates, as he did at the Columbus Am. Soc. C.E. meeting, that the river will accommodate itself to the flow put through it, filling in or excavating just so much as is necessary. However, he does feel that information is not available to prove or disprove his theory. He is one of those engineers who would not object to putting the whole Mississippi down the Atchafalaya, converting the present channel into an arm of the Gulf. Since in that event no silt would come down, the channel would remain as at present 150 to 200 ft. deep opposite New Orleans, ample for any ship afloat. So convinced of the efficacy of this plan is Mr. Coleman that he has finally persuaded the War Department to let him try it out on a certain river in Texas emptying into the Gulf. Observation of this scheme in operation for several years he believes should be invaluable in solving the Mississippi outlet problem.



DRAINAGE SYSTEM OF NEW ORLEANS SHOWING PUMPING
All water that falls in New Orleans must be pumped out through the levees to Lake Pontchartrain, which is normally 0.8 ft. above mean gulf level.

For years I have had a desire to see data on the quantity of sediment borne by the Mississippi as it passes New Orleans. A study of carrying capacity, fineness of material, the physics and hydraulics of floating or suspended matter under various velocities and conditions, it has seemed to me, was fundamental to any attempt to answer the question of how serious would be the clogging of the river below spillways. Col. Garsaud says the effect will be nil and Mr. Coleman sees the river closed.

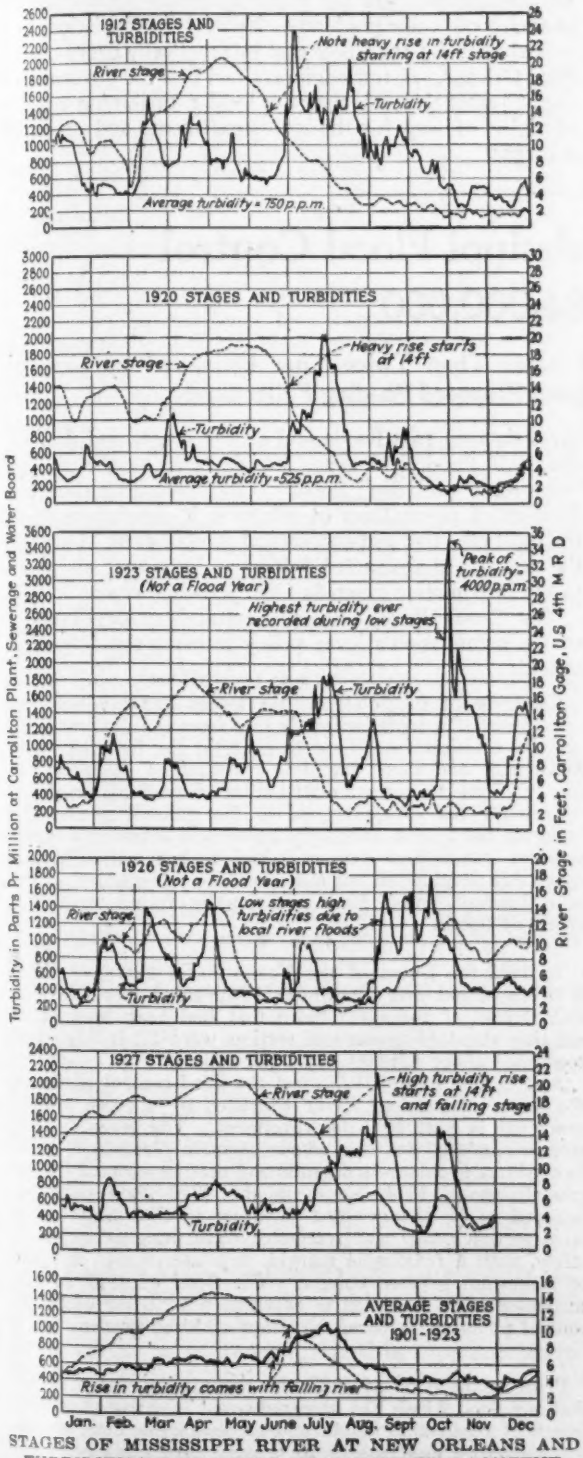
Observations below the Poydras Crevasse in 1922 were to have been provided for by the engineers of New Orleans but a misunderstanding as to their observations being duplicated by the Mississippi River Commission resulted in the information not being obtained. However, I did find that Mr. Earl has compiled certain information bearing on the point as far back as 1901 in the way of charts on which are plotted river stages and turbidities. From these charts, certain ones of which are reproduced herewith, one easily comes to the conclusion that the water at the higher stages does not carry the higher turbidities. The rise in amount of sediment comes with a falling gage and is due no doubt to caving banks. Mr. Earl is a stanch advocate of controlled spillways and he argues from studies of these charts that the sills should be near 14 ft. on the Carrolton gage or if placed lower that the outflow be cut off at the 14-ft. stage, since at that time turbidities most often begin to rise rapidly.

It is studies of turbidity and carrying capacity, with sediment content readings taken at many points in the river cross section, together with velocity observations, that need to be made. The observations of Humphrey & Abbot are good, but we need more. These opinions are set down as my own, but they are borne out by many engineers in New Orleans.

Since writing the above there has come to my attention a paper, recently presented to the Illinois Chamber of Commerce by Col. Edward H. Schulz, district engineer

in charge of the U. S. engineer office in Chicago, in which he gives certain general figures on sediment as being 1/1,500 by weight and 1/2,900 by volume of the discharge varying according to velocity, character of beds and banks, whirls and eddies.

Since scouring velocities in front of New Orleans at the 1927 stages with the crevasse at Caernarvon open were obtained it is evident that a spillway at that point


1912 STAGES AND TURBIDITIES
Note heavy rise in turbidity starting at 14ft stage
River stage
Turbidity
Average turbidity = 750 p.p.m.
1920 STAGES AND TURBIDITIES
Heavy rise starts at 14ft
River stage
Turbidity
Average turbidity = 525 p.p.m.
1923 STAGES AND TURBIDITIES (Not a Flood Year)
Peak of turbidity = 4000 p.p.m.
Highest turbidity ever recorded during low stages
River stage
Turbidity
1926 STAGES AND TURBIDITIES (Not a Flood Year)
Low stages high turbidities due to local river floods
River stage
Turbidity
1927 STAGES AND TURBIDITIES
River stage
High turbidity rise starts at 14ft and falling stage
Turbidity
AVERAGE STAGES AND TURBIDITIES 1901-1923
Rise in turbidity comes with falling river
Jan. Feb. Mar Apr May June July Aug. Sept Oct Nov Dec
Turbidity in Parts Pr Million at Carrollton Plant, Sewerage and Water Board
River Stage in Feet, Carrollton Gage, U.S. 4th M R D

STAGES OF MISSISSIPPI RIVER AT NEW ORLEANS AND TURBIDITIES INDICATING THAT SEDIMENT CONTENT RISES AFTER FLOOD STAGES ARE PAST

would not be expedient even though it did cut the height of gage at Carrolton by 2 ft. It would have to be accompanied by a second spillway above town, probably at Bonnet Carré, to maintain a flatter slope by the city. This is the reason why two spillways are always referred to as being necessary at New Orleans.

Revetment Protection Needed

Revetments—A spillway below New Orleans is dangerous mainly because of the scour the increased slope and induced velocity produces. Docks were damaged, depths of 35 ft. were increased to 50 ft. and at two places in front of the city revetments have had to be repaired, all as the result of the Caernarvon Crevasse, which Col. Garsaud states increased the velocities up to 8½ ft. per second.

Revetments are 5 to 10 times as expensive as levees and the maintenance is 10 per cent per year, said Maj. W. H. Holcombe, U. S. engineer in charge of the Fourth District, which extends from Vicksburg to the Gulf. Whether to set back the levees behind a caving bank or revet the point of scour are alternates necessary to figure. Property values enter largely into the cost in the area near New Orleans. Usually the Mississippi River Commission sets back a levee to last at least 20 years. Revetment in the lower river is a heavy item because the river is well fixed in its course, senile, it is called. There are 127 miles of revetment now and Col. Schulz estimates the cost at $300,000 to $400,000 per mile. Major Holcombe's figure of $70 per foot checks this figure. Expenditures for revetment in the Fourth District in consequence run extremely high.

The New Orleans Dock Board is continually applying to the Mississippi River Commission for more revetments but appropriations will go only so far and nowhere near 100 per cent safety from revetments has been obtained. Several hundred miles additional revetment are needed in the whole river.

Special machines for making and laying the concrete-slab type of revetment have been devised by engineers of the commission. In one the slabs are articulated but in the latest machine, which it is hoped will be described in *Engineering News-Record* by the designer at a later date, the slabs are laid like huge shingles. This outfit is now at work in the Third District near Hickman, Ky.

On the point of scour and deposition, Col. Garsaud is of the opinion that floods have much less effect on the regimen of rivers than ordinarily attributed to them. It is some flow below bank-full stage which occurs for so much longer periods of time that exerts the greatest amount of power and really determines the river's normal regimen, in his opinion.

Hydraulic Studies Needed

Hydraulic Interference—The hydraulics of interferences due to the entrance or exit of water from the main body of the river seems not to be understood. It was studied at Cairo this year and at the entrance of the Arkansas. Much higher heads were built up than could be accounted for by ordinary hydraulic formulas. Mr. Klorer has tried to interest hydraulic laboratories of universities in the phenomenon but without success so far. What actually happens to the heads above and below crevasses needs study also, in his opinion.

For New Orleans one concludes from these interviews that safety lies in strengthening and raising her back-door levees and repairing and adding much revetment to

those at her front door as a present-day job. Eventually spillways above and below the city may be decided upon as giving the greatest measure of safety but first a lot of engineers want to be sure what abstraction of say 250,000 sec.-ft. of water will do to the river. Fortunately the Federal Spillway Board was still in session during the 1927 flood and its findings should be the last word. Finally the question of how much water is to be turned down the Atchafalaya, whether controlled or not, and how best to provide the necessary floodway for this new river which is less than 60 years of age. Men are still living who have walked a log across it when it was but a chute. That this part of the problem of safety for the lower river cannot be arrived at without detailed surveys of terrain, and a close survey of land use seems obvious. How what New Orleans wants will fit in with the requirements upstream is something that must be considered. Col. Garsaud estimated $106,000,000 is necessary for the Atchafalaya floodway, Bonnet Carré and Caernarvon spillways and levee strengthening. Since much in the estimate is for land purchase, damage or easement, he has recently been figuring on what can be done with $50,000,000. Not until the upper levees are strengthened are the lower spillways needed so the financing can come gradually. Federal appropriations are looked upon for the whole cost by practically every one in New Orleans. A few engineers feel that the government might reasonably ask that the local contribution consist of rights of way for the levees easements and property damages.

Commission Finds Mississippi Flood Control To Cost $684,000,000

Recommends Comprehensive Control Project At Cost More Than Twice That Estimated by General Jadwin—Reduced Working Program Proposed Pending Fuller Study

COMPLETE control of Mississippi River floods is estimated by the Mississippi River Commission to cost $684,000,000 without land cost and damages, or more than twice as much as stated by Maj.-Gen. Edgar Jadwin, Chief of Engineers, U. S. Army, in his report summarized last week. The two projects are closely the same (i.e. they comprise leveee raising and strengthening large flood bypasses through the Tensas and Atchafalaya basins, a spillway to tidewater just above New Orleans, and thoroughgoing revetment work to stabilize the river) except that the commission recommends (1) an additional spillway at Caernarvon, just below New Orleans and (2) provision for control spillways at the heads of the two west-bank diversion channels, while it omits the New Madrid floodway which Gen. Jadwin proposes. Further, the commission says that "certain features of this comprehensive plan require further investigation," and it therefore recommends for immediate execution a reduced project of essential items at a cost of $317,500,000 exclusive of land and damages, a figure slightly greater than the cost of Gen. Jadwin's full project.

The Commission's figures are:

	Comprehensive Project		Working Project	
Levees		$410,000,000		$150,000,000
Diversions				
Cypress Creek	$107,000,000		$170,000,000	
Atchafalaya	52,500,000		52,000,000	
Bonnet Carré	11,500,000		11,500,000	171,500,000
Caernarvon	10,000,000	181,000,000		
Revetment		165,000,000		75,000,000
Dredging		7,000,000		3,000,000
Supervision, etc		12,000,000		8,000,000
Total		$775,000,000		$407,500,000

These totals include respectively $91,000,000 and $90,000,000 for rights of way and other damages. The commission also recommends an economic survey of the region to determine allowable flood-protection expenditure.

In reporting on its investigation, which was made in response to a request of the Chief of Engineers dated May 4, the commission reviews briefly the history of Mississippi River improvement work since the first New Orleans levees were built in 1717-1727, and the plans adopted by it and its predecessors prior to 1927. It then gives data on past floods and on the stages and flow of the 1927 flood, and states its views as to the size of "maximum probable" flood. After analyzing different flood-relief possibilities or schemes, it determines upon the most effective and economical combination. Parts of the report are abstracted below, as important additions to the information given in Gen. Jadwin's report.

The Levee Policy—On the cut-and-try development of the commission's levee policy hitherto the following is said:

Confinement of floods within levees has involved the estimation of a tentative grade line from the data at hand, and its revision from time to time, in the light of fuller knowledge and more complete restraint of overflow. It has involved the careful distribution of available resources to provide the greatest good to the greatest number by bringing the system uniformly up toward temporary standards of grade and section. And it has naturally and inevitably involved a full measure of condemnation as the still deficient levees have occasionally failed under the stress of floods.

* * *

In 1882 the levees of the Yazoo Basin averaged about 8 ft. in height and contained 31,500 cu.yd. per mile. In 1927 levees of the same basin that had been brought to existing standard grade and section were 22 ft. high and contained about 421,000 cu.yd. per mile.

At the time of the 1927 flood the flood-control works of the lower Mississippi River consisted of a levee system protected in part by bank revetment. The levee system, still incomplete, was one whose ultimate dimensions were to conform to a standard grade and section adopted by the commission in 1914, based on the 1912 flood confined, believed to be the greatest flood up to that time. The cross-section grade and freeboard were designed to carry safely, with a reasonable margin, any later floods of equal or somewhat greater volume. The flood of 1927 so far surpassed prior floods as to necessitate a review of flood-control plans and a reconsideration of levee grades.

* * *

Prior to the flood of 1927, the commission was satisfied that any flood which had occurred could be carried between levees of reasonable height and at reasonable cost, and it was therefore unnecessary to provide diversions or other auxiliary methods of relief. It is now demonstrated that such auxiliaries should receive due consideration in any

flood-control project designed to carry such a flood as that of 1927.... To provide protection against the assumed probable maximum flood of the future will require large increases in levee heights or a resort to auxiliary means of relief. Comparative estimates of cost indicate that in many places a combination of levees and spillways . . . will be cheaper and give greater safety than levees alone.

Maximum Probable Flood—Discharge figures for the 1927 flood in comparison with some large prior floods are given by the commission in a number of tables, from which the following is condensed:

MISSISSIPPI RIVER FLOOD FLOWS, IN SEC.-FT.

Year	At Columbus	At Arkansas City	Carried to Basins Above Old River	Outflow Below Old River
1892	1,401,000	1,742,000	1,960,000	1,685,000
1912	2,015,000	2,007,000	1,927,000	1,912,000
1913	2,015,000	1,782,000	1,864,000	1,701,000
1916	1,775,000	1,889,000	1,926,000	1,712,000
1922	1,501,000	1,761,000	1,946,000	1,696,000
1927	1,728,000	1,712,000	2,010,000	2,053,000
1927 confined		2,472,000		2,336,000
(estimated) (Max. Prob.)	(2,250,000) (at Cairo)	(2,850,000)		(2,650,000)

The last line of this table gives the flood which the commission believes "may be conservatively assumed as the probable maximum flood of the future," a flood about 25 per cent greater than that of 1927. On this future flood the project and studies of the commission are based.

In arriving at this future flood, the commission says that (a) if the greatest recorded discharges of the Wabash, Ohio, Cumberland, Tennessee and Upper Mississippi are combined at Cairo, a flow of more than 3,000,000 sec.-ft. is obtained; (b) the maximum of the Ohio and the Upper Mississippi combined give 2,700,000; and (c) the confined 1927 discharge increased by the excess of Ohio flow in 1913 over 1927 would be 2,400,000 sec.-ft.

"All these combinations may be classed as possibilities, but it does not seem probable that rainfall sufficient to produce such coincident floods will ever occur. In the opinion of the commission the maximum probable flood of the future should be assumed as one in excess of that of 1927 but less than the combination set up in (c)."

The method of deriving the figures for Arkansas City and Old River is not described.

Levee Construction—The report says that "the cross-section of the levees should be strengthened and certain parts of the present grade line, since shown to be low, should be raised."

The standard levee section of the last 13 years has a crown width of 8 ft., a 3-ft. freeboard above 1912 confined flood grade, a riverside slope of 1 on 3, a landside slope of 1 on 3 in the upper part, a banquette 5 to 8 ft. below crown and 20 to 40 ft. wide according to height of levee, and a slope of 1 on 4 below the banquette. Based on experience with this section and to provide a larger factor of safety, the following section is now recommended for use below Cape Girardeau: Crown width 12 ft., 5 ft. freeboard (but below New Orleans decreasing to 3 ft. at Fort Jackson), riverside slope 1 on 4, landside slope (no banquette) 1 on 4 to 1 on 6½, the section to contain completely a 1 on 7 saturation line starting on the river slope 2 ft. below crown. Where a construction road is needed it should be superimposed on the landside slope.

"These sections and freeboard are much greater than heretofore considered, but the commission believes that in any comprehensive plan no smaller section should be adopted."

In the reduced working plan recommended for present execution, this same section is used but the old 3 ft. freeboard is retained.

Levees Only—The levee grade line computed for the "maximum probable" flood wholly held between levees, with 5 ft. freeboard, would be above present crown 12.4 ft. at Cairo, 6.9 ft. at Memphis, 12.5 ft. at Helena, 10.0 ft. at Arkansas City, 14.0 ft. at Vicksburg, 15.2 ft. at Natchez, 12.0 ft. at Red River landing, 9.8 ft. at Baton Rouge, 5.6 ft. at Carrollton (New Orleans) and 3.4 ft. at Fort Jackson. To raise the levees accordingly, with the increased section recommended, would cost $566,500,000 for the main river and $115,500,000 for the tributaries (partial only), or a total of $682,000,000 not counting revetment and other work. As already quoted, the commission finds a combination of levees with spillways and bypasses cheaper and safer.

Diversion of Tributaries—Diversion of the Tennessee River to the Tombigbee by way of Bear Creek was found to cost $348,000,000 for 88,000 sec.-ft., lowering the Cairo stage 1½ ft., and hence "is deemed impracticable." Diversion of the Red River to the Gulf through Mermentau River, the cheapest of several possibilities, would cost $11,700,000 for 300,000 sec.-ft., which is much higher than the cost of the Atchafalaya bypass, and therefore "is deemed unworthy of further consideration."

Bypasses—Bypasses through the St. Francis basin and the Tensas basin, and diversion of part of the lower river through the Atchafalaya outlet, was studied. As to the upper one, "No diversion channel from Cape Girardeau or vicinity to any other point on the Mississippi River at or above the Arkansas River seems at present feasible, as equal or greater protection can be obtained by levees at decidedly less cost."

The best diversion here would cost $337,600,000 and would save only $27,000,000 in levee cost. But further study of river control in this reach is believed to be advantageous.

The Tensas basin bypass would cost $107,000,000 for 600,000 sec.-ft. and $125,000,000 for 750,000 sec.-ft. It would follow Boggy Bayou, Bayou Macon, Boeuf River, the Ouachita River from Columbia, and Sicily Island gorge to the backwater area of the Tensas basin. If cleared, it would be 2 to 3 miles wide, have a flood slope of 0.29 to 0.65 ft. per mile, and develop a velocity of 1.77 to 2.9 ft. per sec. If uncleared, it would be 5½ to 10 miles wide, have a slope of 0.20 to 1.70 ft. per mile, and develop a velocity of 0.72 to 1.30 ft. per second. Some 44 miles of railroad and 26 miles of highway would have to be elevated. Its effect would be to reduce the flood height at Arkansas City by 8 ft., and to save $82,500,000 over levees alone.

Coming to the Atchafalaya diversion, the commission says that carrying the maximum flood of 2,650,000 sec.ft. down the main river would require both enlargement and relocation of the levees, at a cost of $128,000,000 exclusive of right of way, and exclusive of the cost of protecting New Orleans (estimated at $33,000,000) and Morgan City. The river capacity at New Orleans is given as 1,400,000 sec.-ft., as a reasonable maximum. By setting back the Atchafalaya levees a flood bypass of 900,000 to 1,000,000 sec.-ft. can be developed; and by providing two controlled spillways on the east bank of the Mississippi, respectively above and below New Orleans, each of 250,000 sec.-ft. capacity, the remainder of the maximum flood flow can be accommodated. The latter would be at Bonnet Carré, above New Orleans, discharging into Lake

Pontchartrain, and at Caernarvon, below New Orleans, discharging into Breton Sound. This system requires some enlargement of the levees at the head of the Atchafalaya basin. It would cost $127,500,000, of which about $53,000,000 each would be required for the Atchafalaya bypass and the levees of the Mississippi; and about $10,000,000 each for the two spillways. The hydraulic features of the Atchafalaya bypass are not described.

Other Means of Flood Protection—"Safety-valve spillways" along the levees, 50 to 80 miles apart, with average discharging capacity of 90,000 sec.-ft. into the basins, were studied and are thought "worthy of further consideration as part of a comprehensive flood-control plan." They would minimize disaster in case of a "superflood" which might come some day. A 5,200-ft. spillway for 3-ft. depth of flow would cost about $2,550,000.

Reservoirs were considered by the commission, using the studies of the separate Reservoir Board. Reservoirs above Cairo, says the commission, would cost far more than equal protection by levees and bypasses. Reservoirs in the basins themselves also appear to be excessively expensive but are considered worthy of further study.

Reforesting is held to be not efficacious, economically unsound, and far too slow in any event. Increase of river capacity by dredging to greater depth is considered, but rejected as impracticable and prohibitively costly. Cutoffs also are considered unsound:

The commission adheres to its policy of preserving the river generally in its present form, and cannot subscribe to a plan of flood control or of improvement for navigation that involves the formation of cutoffs. Rather, the commission believes that its first duty to navigation and to flood control is to prevent cutoffs.

Local Interests Should Share Costs

Cost Allocation—Discussing the subject of cost allocation with respect to the bearing of local contributions upon its work, the commission says that "some degree of local cooperation is essential to successful accomplishment of a flood-control project."

The commission would view with deep concern the adoption of a federal flood-control project that would absolve local interests from participation in costs in levee maintenance. It believes that part of the cost thereof should be borne by local beneficiaries. On the other hand, it believes that the federal government should pay part of the maintenance costs and should reserve full control of such work. The federal government alone is equipped with vessels and plant to meet emergencies.

The commission believes that relocation of the levee line for the purpose of increasing the flood way along the main river may properly be paid for by the United States, except that local levee districts should pay costs of land and right of way and damages.

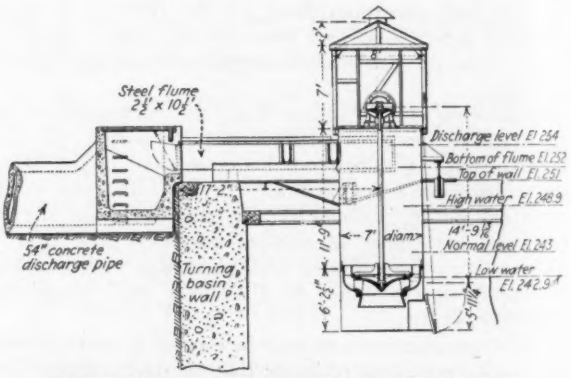
It would have the United States pay for all of the bank protection on the main river and two-thirds of that on tributaries; for all enlargement levees over 1914 grade and section; for relocation of levees to increase the flow area (except cost of land and damages, which should be paid by local levee districts); and for some part of the levee maintenance and repair.

Fuller Study Demanded—The commission states that it has enough data to show that the cost of the proposed work is justified, but it "is not in possession of the data on which to base a complete economic study of the flood-control project.... While the field investigations and further studies contemplated in this report are under way there should also be in progress an economic survey for the purpose of determining the economic limit of flood-protection cost."

Flushing-Pump Installation Clears a Toronto Bay of Pollution

A STAGNANT, unhealthy condition in Ashbridge's Bay, a body of water one-half square mile in area on the east side of Toronto, has been eliminated by means of a pumping installation which supplies fresh water to the bay. The bay has only one outlet into Lake Ontario, another into Toronto Bay having been closed when the Toronto harbor front was reconstructed. The only movement through the remaining small outlet was by means of wind currents and was quite inadequate to remove the pollution. The western end of Ashbridge's Bay is within approximately one-half mile of the city's new turning basin and the pumping plant has been installed there. The following information is abstrated from an article in the *Contract Record and Engineering Review* by George E. Fox, chief engineer, John Inglis Co., Toronto, the manufacturer of the pumps.

From a structural steel frame erected across a corner of the concrete turning basin wall were suspended two



ELEVATION OF ONE OF THE ASHBRIDGE BAY FLUSHING PUMPS

cylindrical tanks, each 7 ft. in diameter and 17 ft. deep, the bottoms of which were submerged 8 ft. below water level. In each tank there was mounted a centrifugal pump having a capacity of 39 m.g.d. against a total head of 9 ft. The design of the pumps is unusual in that the casing has an annular opening on the upper side to discharge the water into the tank above the pump and cause it to rise and flow through the discharge flume. The impellers are of the standard closed vertical centrifugal pump type, mounted on vertical shafts. Each impeller is driven by a 75 hp. 750 r.p.m. horizontal motor through a bevel gear reduction.

Each steel cylinder has a flap gate below the pump which can be closed when it is desired to pump the cylinders dry in order to get access to the pump. Two steel discharge flumes, each 2½ ft. deep and 10½ ft. wide, connected to the tanks above the turning basin wall, discharge into a concrete basin, forming the entry to a 54 in. diameter concrete pipe laid on a gradual grade to the west end of Ashbridge's Bay. The pump house is mounted directly on the tank tops and supported at the ends by concrete piers from the turning basin wall.

The type of pumping equipment chosen for this work was decided on after investigation of a duplicate plant put in by the same company at the Holland River Marsh where 7,500 acres of marsh land was reclaimed by pumping down the old river 5 ft. below its normal level, after digging a canal and damming off the old river.

Southern Pacific Railway of Mexico Is Completed

Heavy-Grade 103-Mile Section From Coast Lands to Interior Plateau Is Last Link in 1,480-Mile Route to City of Mexico—American Engineers and Contractors

THE recent completion of the Southern Pacific Railway of Mexico, connecting with the National Railways of Mexico and giving a new west-coast route from the United States to the City of Mexico, makes it appropriate to present some account of this important line and the vicissitudes of its construction. The general route is shown in Fig. 1, and a plan and profile of the last and most difficult section, ascending from the coast to the interior of the country, are given in Figs. 2 and 3.

The first portion is the old Sonora Railway, from the frontier town of Nogales, Ariz., to the port of Guaymas, on the west coast of Mexico. This line, 265 miles long, was built several years ago and was later purchased by the Southern Pacific Railway of Mexico, which is the Mexican subsidiary of the Southern Pacific Company. From Nogales to Guadalajara, where the new line connects with the National Railways of Mexico, the distance is 1,102 miles. With 380 miles of this latter system, the total distance from Nogales to the City of Mexico is 1,480 miles.

Original Construction—Work on the line from Empalme, about 5 miles north of Guaymas, to Guadalajara was started in 1905 and was to include some 500 miles of branches. In 1912 it had reached Tepic, 671 miles from Empalme; in addition about 45 miles had been built between Orendain and La Quemada, at the southern end. Some branches had also been built and a line started from Nacozari, Mexico, toward the Tonichi branch, but track was never laid on the latter line. There remained a gap of 103 miles between Tepic and La Quemada.

Difficult Location—In closing this gap, the line had to ascend from the coast lands to the central plateau. Railway connection between the interior and the west coast had been desired by the Mexican government for many years. A location survey from Guadalajara to the port of San Blas was made by the Mexican Central Railway about 1882. This line was built from Guadalajara west as far as San Marcos, 50 miles, and for about 12 miles eastward from San Blas; but this latter part of the line, from San Blas, was abandoned and after a few years the rails were removed.

A route descending through the heavy mountainous country of the Barrancas, 55 miles from Guadalajara, had been located by the late A. M. Wellington, but was given up as impracticable. According to an official statement, this location used the Pajaritos Pass, about 7 miles west of the present constructed line, but the distance, grades, curvature and cost involved were so great that this early location received only a general examination by the locatiing staff when the Southern Pacific Railway of Mexico was preparing for construction.

It was concluded, however, that the only practical line from an operating standpoint would have to be built through the Barrancas. Four separate locations were made, each with various revisions and with grades from 1 to 1½ per cent. With the several locations and topographic surveys linked together, a final definite location was adopted. From the profile, Fig. 3, it will be seen that the maximum grades are 1.5 per cent, as against 2.4 and 1.6 per cent on the previously adopted location and that several old proposed grades of 1.5 per cent are replaced by 1 per cent grades.

Revolutionary Troubles—Between 1910 and 1920 the Southern Pacific Railway of Mexico had suffered severely from the revolutionary activities, both by destruction of property, with consequent loss of operating revenue, and by deterioration of property due to lack of adequate maintenance. But completing the railway to a connection with the interior markets of the country was essential to develop sufficient traffic to pay for operation. In 1922, therefore, after two years of quiet, the railroad opened negotiations with the Mexican government toward the settlement of claims for damages.

The agreement made with President Obregon in 1923 extended the concession on account of time lost during the revolution, and provided for prompt closing of the gap between Tepic and La Quemada. It provided also that the money received for damages should be applied in rehabilitating the main line and in reconstruction of the Tonichi and Alamos branches, 100 and 37 miles respectively, which had been out of commission for some years. On March 6, 1923, President Obregon turned the first spade of work on the new line.

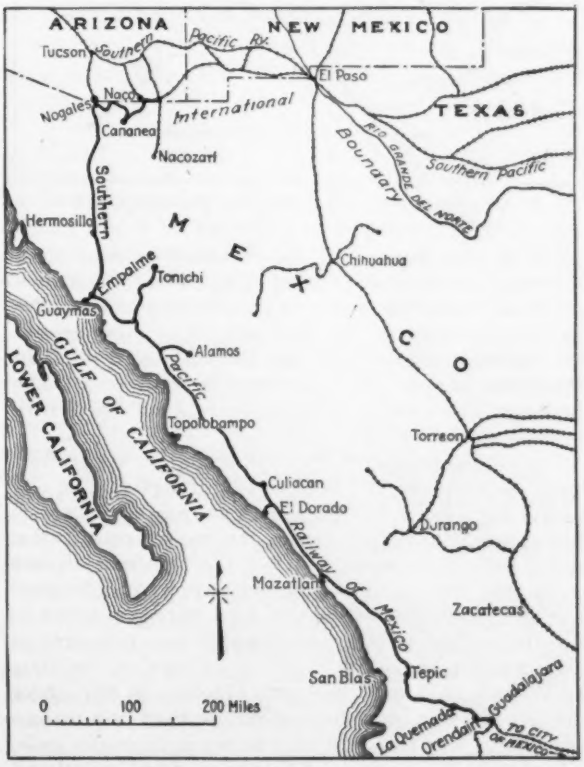


FIG. 1—SOUTHERN PACIFIC RAILWAY OF MEXICO

Construction Resumed—Four locating parties were placed in the field. As the previous surveys between Tepic and La Quemada had been completed in 1910, all stakes and bench marks had disappeared and lines cleared through brush country had been grown over. Attempts to work from the original adopted location were finally

abandoned. Furthermore, tentative studies of the records indicated that at certain points lighter grades could be introduced at relatively little additional cost, in view of the reduced operating expenses.

Construction of the first fill at La Quemada was started in 1923 with basket men, or piece-work laborers, who carry the excavated material in baskets secured by a tumpline around the forehead. A contract was then let to W. W. Burns, who completed about 4½ miles, partly on cost-plus and partly on unit-cost basis. In March the first 10 miles south of Tepic was let to Twohy Brothers. Finally, in May, a contract for the remainder of the line, about 88 miles, was awarded to the Utah Construction Company.

As the rainy season starts in June and makes the

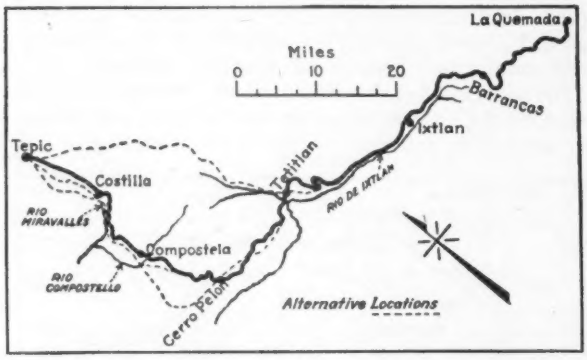


FIG. 2—MAP OF TEPIC AND LA QUEMADA LINK

native roads impassable for wheeled vehicles until November, the contractors had to repair old roads, build new roads, establish camps, haul equipment and do other preliminary work at the beginning of a rainy season. In addition, the road from the end of steel crossed the Miravalles canyon about 30 times in 7 miles. Under these conditions active construction was not commenced until November.

The original plan of the work was to concentrate forces between the rail head and the Barrancas, leaving a small force in the Barrancas to build wagon roads; and then to concentrate men and machinery for construction in this difficult section. To avoid the high cost of such roads, this plan was changed and men were distributed over the whole line. However, men were not found so plentiful as had been expected and it was necessary to make another change in construction method, building truck roads on a high line with branches to the longer tunnels. Then in order to complete the work in a reasonable time it was also necessary to resort to power tools.

Difficulties Encountered—Numerous troubles delayed progress. During the Huerta revolution, which broke out in December, 1923, the federal troops controlled part of the line and the remainder was in the hands of the rebels. By interviews with the opposing commanders an agreement was made that the railroad forces would not be molested, hence this trouble caused little delay. In August, 1924, a slide at the south end of tunnel No. 4 blocked this tunnel for five months and largely interrupted the handling of heavy supplies to the construction front.

In January, 1926, communication from the north was cut off by floods which destroyed several bridges on the operated part of the line. Then in September the burning of bridges by Yaqui Indians was followed by a disastrous flood that paralyzed operations for about six weeks. The rainy season was unusually severe where the line was under construction, more than 100 in. of rainfall being recorded at Tepic between November, 1925, and November, 1926.

Tunnel Work—The 31 tunnels aggregate 24,215 ft. in length; the two longest are 2,880 ft. each. Native timber, mostly scrub pine, was permitted for full-section lining in several tunnels in the Barrancas district, with the understanding that this timbering was to be followed promptly by the standard reinforced-concrete lining. In several cases, however, this final lining was delayed. Native timber was used in pioneer headings and trap drifts. Some 8x12-in. timbering in other tunnels was afterward embedded in the concrete. Redwood timbers for permanent or semi-permanent lining were 10x14 in. and from 12x12 in. to 12x24 in. Both three- and five-segment arches were used, the latter where wall plates were required.

Most of the shorter tunnels were driven with hand tools. Compressed air from portable gasoline plants and three oil-engine plants was used at twelve tunnels. Heavy and difficult ground was encountered in nine tunnels, causing more or less serious caving. In April, 1926, the ground above tunnel 12 showed signs of settling, due to failure of the native timber of the lining. Caving or slides at two places blocked the tunnel for several months. As it was important to complete track-laying beyond the point in order to start erection on the Salsipuedes viaduct, a shoofly track was built, with 12-deg. (metric) curves. This tunnel was lined eventually with reinforced concrete 20 to 36 in. thick.

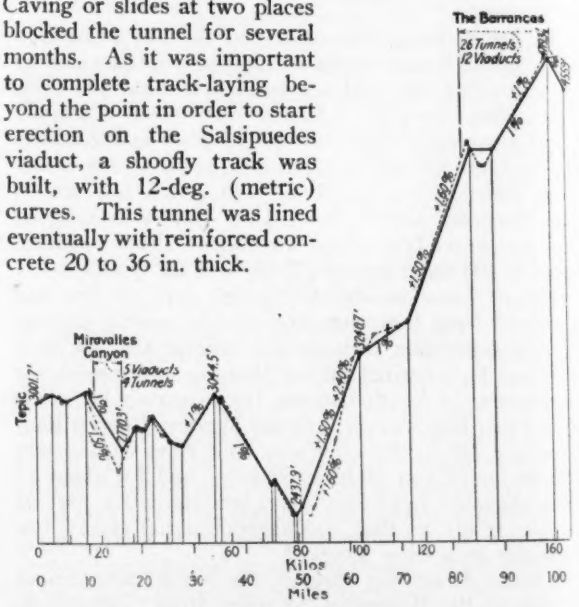


FIG. 3—PROFILE OF HEAVY-GRADE SECTION

At tunnel 18 in July, 1925, a slide started on the east side of the north portal about 100 ft. above the crown. Heavy timbering was required to hold the excessive pressure on this side. By April, 1926, the ground seemed to have stabilized and a reinforced-concrete lining, with invert, was placed. All timbering was removed. During the concreting, sections 8 ft. long were left unsupported for three days without showing settlement. But in August cracks were noted in the concrete on the east side near the north portal. The failure started near the floor, extending south and upward about 45 deg. and finally affecting about 70 ft. of lining.

Samples of concrete from the wall near the failure were porous and had no bond with the steel. Stalactites had formed on the arch where water had come through,

and brown stains on walls and roof indicated steel corrosion. Laboratory tests made in San Francisco indicated that the concrete had been mixed and placed properly, but that the water contained sufficient hydrogen-sulphide gas to break down the concrete either during or after setting. Waterproofing was recommended, but as conditions made it impossible to apply effective waterproofing, especially as an invert was required, the failed section was replaced with a solid lining of 12x24-in. redwood timbers. Similar failure of a short section of tunnel 14 was remedied in the same way.

Excavation—The yardage of open excavation had to be increased to about 25 per cent over the preliminary estimates owing to the instability of the material. It was necessary to flatten the slopes in cuts, while numerous line changes were made on account of material not making stable fills. The greater part of this work was done with hand tools on piece-work basis, a specific number of wheelbarrow, mine-car or mule-cart loads being assigned per man or per shift, according to the length of haul.

TRACKLAYING PROGRESS FROM TEPIC

To Costilla	May 10, 1923	Ixtlan*	Aug. 24, 1925
Tunnel No. 2	Dec. 17, 1924	(71 miles)	
Tunnel No. 3	Jan. 8, 1925	Tunnel No. 6	March 28, 1926
Miravalles Canyon	Feb. 24, 1925	Barrancas	June 3, 1926
Compostela River	March 12, 1925	(80 miles)	
(20 miles)		Salsipuedes viaduct†	Jan. 27, 1927
Cerro Pelon	May 1, 1925		
Tetitlan	June 18, 1925		

*Tracklaying held up at Ixtlan from Aug. 24 to Dec. 15, 1925, on account of slide at tunnel No. 4, which delayed forwarding of track material.
†Tracklaying north from La Quemada and reached the Salsipuedes viaduct a few days before.

Structures—There are 32 steel bridges, 20 to 860 ft. long, the most important being the Salsipuedes viaduct, 860 ft. long with its rail level 240 ft. above bed of stream. Steel erection was done partly by the Utah Construction Company, which sublet the remainder of this work to the United States Steel Products Company. On the rubble and ashlar masonry, used in large quantities, the workmanship was especially good and of pleasing appearance.

Tracklaying—In general, the line was laid with 65-lb. rails; but 75-lb. rails were used in the Miravalles canyon and through the Barrancas, and also on all curves of more than 3 deg. (metric). Progress on the tracklaying from Tepic, which followed close behind the grading, was as shown in the accompanying table (see also Fig. 2).

Shoofly Tracks—In the reconstruction of the older part of the railway, which had suffered from numerous bandit raids, shoofly tracks around damaged structures were employed almost entirely, as the line was not adequately patrolled to insure safe operation over timber cribwork. No tables or instruments were used by field forces in making these shooflies, and grades were held to approximately 3 per cent, dropping down into and coming out of the shooflies. Some of these shooflies were so deep and so long that they had to be maintained for speeds of 25 to 30 m.p.h. in order to get the trains out of them.

In most cases where it was necessary to figure on operation through waterways, the elevation for the top of the track ties was established practically level with the bed of the stream. For support a second tie was placed below the track tie and fastened to it by means of drift bolts, the earth or other material from the stream bed being removed and heavy rock placed by hand between the ties and to about 3 ft. in width along the ends of the ties on the downstream side. With the track anchored in this way, the waterway was not obstructed, while scour was prevented by the rock apron. Little work was necessary to put these points in shape after a freshet.

Engineers and Contractors—For the Utah Construction Company, the work was in charge of E. O. Wattis, H. J. Lawler and L. S. Corey, with J. Q. Barlow as consulting engineer. For the railroad company all work was under the direction of E. B. Sloan, chief engineer of the Southern Pacific Railway of Mexico. W. I. Bassett was chief locating engineer, with J. C. Galbreath, L. V. Gillette, F. O. Pease, G. B. Little and H. A. Ayres locating engineers. The construction organization was headed by L. V. Gillette, assistant to chief engineer, with R. Castaneda in charge of the work from La Quemada northward. Resident engineers included S. J. Wright, G. B. Little, G. C. Knight, F. M. Townsend, Julius Irion, J. S. Culbertson, B. A. Savage, D. F. Giboney, W. J. Winslow, C. W. Moberg and H. A. Ayres. The railway was opened to traffic in April, 1927.

Interconnected Power Systems Favored

The Federal Power Commission has received a communication from the Secretary of the International Executive Council of the World Power Conference transmitting a resolution passed by the Council at its latest annual meeting in Cernobbio, Italy, on Sept. 7. This resolution which follows, is an appeal to the governments of the various countries to facilitate by all means in their power the operation of interconnected transmission systems, nationally and internationally, and to remove all crippling restrictions and burdens on international transmission of power:

"*WHEREAS*, the Council realises the great benefits to be derived from the extension of the use of electrical energy by the interconnection of transmission systems, nationally and internationally.

"*RESOLVES* to issue an appeal to governments or appropriate national organizations in all countries to facilitate by all the means in their power the operation of such systems; and to remove all crippling restrictions and burdens which may tend to increase the price of electrical energy or to hinder the development of national or international interconnection systems."

The resolution is the result of discussions which took place in the sessions of the World Power Conference at Basle, Switzerland, in 1926, and was proposed by members of the conference from Switzerland, a country which engages extensively in international exchange of power. The resolution has particular application to the situation in Europe where, on account of limited national areas and many examples of international streams, possibilities of electric power interchange across national frontiers are numerous. Its only present application to the United States is in relation to interchanges with Canada.

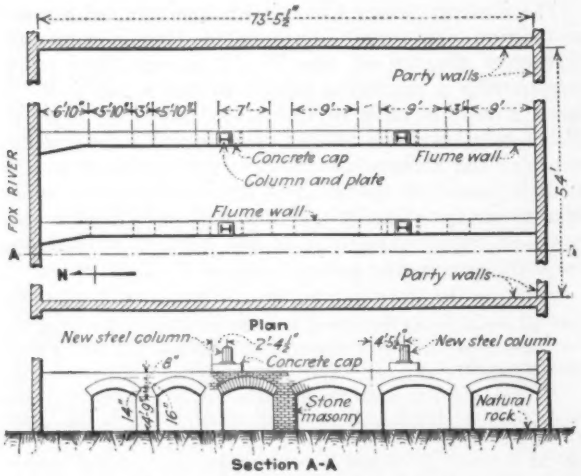
The World Power Conference is an international organization created in 1924, and which held its first general conference at London in that year. It is made up of representatives of 46 countries, and is engaged in promoting the development of power through exchange of information and through discussion of the technical, legal and economic problems of power development, distribution and use. The executive Secretary of the Federal Power Commission is chairman of the American section.

Collapse of Paper Mill Due to Faulty Reconstruction

New Steel Columns Placed Over Arched Openings in Masonry Foundation Walls Instead of Over the Piers

THE disastrous collapse of a building at the mill of the Kimberly-Clark Paper Co., at Kimberly, Wis., was noted in *Engineering News-Record* of Oct. 13, p. 610. Since then the accident has been investigated by the Wisconsin State Industrial Commission, with the following results, substantially as reported by O. T. Nelson, assistant building engineer of the commission.

This paper mill is located on the south bank of the Fox River and its various units are inter-connected. The collapse occurred in a two-story beater-room section about 54x75½ ft., having walls of common brick 17 in. thick, with foundation walls of local limestone. Three of these walls were in common with other units of the plant, the north wall only being an outside wall. The two floors were of reinforced-concrete, each supported by structural steel columns, girders and beams. Four columns 18-ft. long supported the first floor, each consisting of four angles 6x6½-in. and a web plate 15x⅜-in. The two longitudinal rows of girders were built up of four angles 6x6½ in. angles, a web plate 28x⅜-in. and two cover plates 12½x⅜-in., together with stiffener angles and filler plates at points of concentration. Floor beams consisting of 18-in. 54.7-lb. I-beams, at right angles to the girders, were spaced approximately 8 ft. c. to c. The supports for the second floor were similar, but with lighter columns 17 ft. 6 in. long.



FAULTY DESIGN OF PAPER MILL
Note new columns located over arches instead of over piers of old foundation walls.

The first-floor columns were supported by two old flume walls 25 in. wide composed of limestone laid in lime mortar and spaced about 12 ft. apart. These walls, running lengthwise of the building, as shown in the accompanying plan, consisted of a series of arches varying from 5 ft. 10 in. to 9 ft. span, and 5 ft. 11 in. in height to the soffit of the arch. The depth of the arch stone was 16 in., with 8 in. of rubble masonry above, making a total height of foundation walls of 7 ft. 11 in. from the solid rock bearings. The piers supporting the arches were 3 ft. long. This open masonry construction was the original wheel pit and discharge chamber. The heavy flume construction, consisting of timbers 12x12-in. and 14x14-in., together with thick planking, rested on the stone work.

The original building of wood and masonry had been erected about 1890, but in the spring of 1926, the interior portion had been rebuilt with structural steel as described, using only the four outside brick walls and the stone flume walls of the old construction, together with the timber flume, which was left in place. New steel columns had been placed on the old stone masonry without careful investigation of the nature or condition of this foundation. It is reported that these walls were, at that time, immersed in water to a line well over the arch stones, and the walls were assumed to be of solid masonry throughout, instead of a series of arches. Measurements taken after removal of the débris showed that the two north columns had actually been placed 2 ft. 4½ in. off the centers of the piers and 10½ in. on the arch, from the edge of the piers, as shown on the drawing. The two south columns were 4 ft. 5½ in. off the center of the piers, or 2 ft. 11½ in. on the arch, beyond the edges of the piers. Concrete caps 25x5 in. and 12 in. thick had been placed as bearing plates, with each column anchored by four bolts.

At the time of the collapse the first floor was loaded with 3½ carloads of pulp, weighing approximately 297,500 lb., in addition to a beater machine and two beater screens weighing 110,000 lb. The second floor load consisted of 5¼ carloads of pulp weighing 446,250 lb., with two beater machines weighing 160,000 lb. An analysis of the dead and live load indicated a total load of 192 lb. per square foot on the first floor and 252 lb. on the second floor. A check of the steel erection plans indicated that these had been figured for total loads of 250 lb. and 350 lb. respectively. On the basis of these total loads, the actual load transmitted to the rubble-stone foundation walls by each column was 182,040 lb., or 361 lb. per square inch of base plates.

The only portions of these walls that remained intact were the sections between the north wall and the line of the north columns. The remaining piers had either been tipped over or twisted out of place and the arches broken up. In general, the remaining stone work was in good condition, except at the water line where mortar had been washed out to some extent. The effects of erosion were also noticeable in the lower portions of some of the piers. The extent of this damage could not be determined, but no doubt the bearing area was reduced considerably. There did not appear to be any deterioration of masonry through the action of paper mill wastes in the water.

When the collapse occurred, workmen were removing the framework of the old timber flume to permit of placing a reinforced-concrete slab floor on the old foundation walls. This work was being done under the supervision of the company's and contractor's engineers. There was no evidence of any of the masonry having been disturbed by the contractor prior to the failure of the building. It is considered evident that the collapse was caused by the failure of the foundation walls supporting the first floor columns. It is also held reasonable to conclude that the removal of the old timber flume framing either disturbed the equilibrium of the column loads or removed a modicum of support from the arches, which were undoubtedly on the verge of collapse.

The placing of the new interior superstructure on the old stone foundation walls in 1926 was in violation of the State Building Code. All work on public buildings and places of employment in Wisconsin is required to conform to the code, and plans and specifications for such work, except for small buildings, are required to be submitted to the Industrial Commission of Wisconsin for approval. It appears that the engineers of the company considered this work to come under the exception, so that nature of the construction did not come to the attention of the Industrial Commission until after the collapse.

European Advance in Pit Drainage Using Well Points

Preparatory Study and Types of Wells Employed Compared With Well-Point Drainage for Foundation Pit at Lynn, Mass.

Abstract of a paper by Charles Terzaghi on the foundations for the new sewage pumping station at Lynn, Mass., published in the Journal of the Boston Society of Civil Engineers.

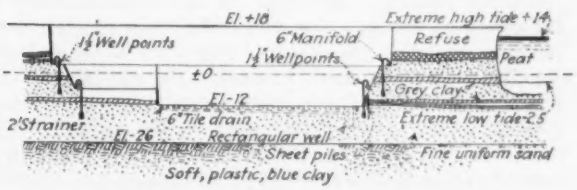
WELL-POINT drainage was successfully employed last spring in excavating a foundation pit 30 ft. deep and about 9,000 sq.ft. in area with underground conditions as indicated by the accompanying sketch. Besides the nearby tidewater, there was to one side of the pit and 55 ft. away a building on piles whose bottoms were 7 or 8 ft. above the proposed pit bottom. The soil below the fill of refuse was fine sand, stable, firm and a good foundation bed on which to work when dry but quicksand when wet; some distance below the proposed subgrade level was clay. In preference to an inclosure of steel sheeting bottoming in the clay, the contractors decided to drain the sand by means of well points and pumping.

After cutting through the refuse, a circumferential ring of wells was formed around the edge of the pit by jetting down 12-ft. pipes with well points, spaced 2 ft. apart. These wells were connected by hose to a 6-in. pipe encircling the pit. They were sunk to about El. —8. Two motor-driven pumps (one a standby) did the pumping. This set of wells lowered the water to about El. —4 or —5 and the sand was taken out dry to El. —2. A second well-point and pumping outfit like the first was then installed, the points going down to El. —14 to —16. Excavation continued in the dry to El. —9, when water, percolating along the seams of clay, accumulated on the bottom. This water was handled to a sump by drains and portions of the pit sides were sheeted to reduce caving.

A 6-in. tile drain at about El. —12 was carried around the pit outside the building line. From this a cross-drain ran to a 3-ft.-square sump in one corner. For the sump a square of sheetpiles was driven and the sand excavated. Inside the sheeted hole a timber frame covered with 4-mesh wire fabric was set, and filled with gravel; then the sheeting was pulled. To lay the tile drains in the running sand a rectangular steel shield was used to hold the trench sides until the tile were in and covered with 9 in. of gravel. This shield was shifted along as the trenching and tile-laying proceeded. With this drainage the excavation continued in the dry to El. —12.

In comment on the methods employed it perhaps deserves notice that European engineers have worked out a whole science, dealing with both the theoretical principles and the empirical side of lowering the ground-water level by pumping from wells. Complete studies of the layout of the wells and of the pumping equipment are made in each individual case. The design is based on the results of pumping tests made on test wells, and it includes accurately computing the position of the lowered ground-water level. During construction careful observations are made (by means of observation wells) of the position of the ground-water level within and outside of the pit, and the quantity of water pumped from the wells is measured. The data thus obtained serve to check the accuracy of the forecast and to detect promptly any abnormal discrepancy between the forecast and the actual conditions.

According to the technique which is used in Europe, the wells, when in fine material, have a diameter of at least 10 in. (instead of 1½ in.). Even for coarse material they are seldom less than 6 in. in size. The distance between the wells ranges between 15 and 50 ft., depending on the permeability of the material. The bottom of the wells extends to a depth of at least 15 ft. below the bottom of the excavation (compared with 2 to 4 ft. in Lynn). The perforated part of the pipe has a length of at least 12 to 17 ft. (compared with 2 ft. at Lynn); and if there are layers of silt or clay in the ground, care is taken that every single intermediate permeable layer is in contact with a perforated pipe section. In Lynn a layer of soil from 4 to 6 ft. thick was practically out of direct communication with the strainers of both the upper and the lower set of pipes—hence the necessity of constructing an auxiliary well within the pit, of driving a row of short steel piles to prevent caving and of draining the bottom by means of tile drains.



FOUNDATION PIT DRAINED BY WELL POINTS AND PUMPING

In Lynn the well points were jetted into the ground. In Europe the wells are constructed in the following manner: By ordinary well drilling one first sinks a casing into the ground, the diameter of which is at least 2 in. larger than the diameter of the strainer. Then the strainer is introduced into the hole, with a plain extension piece reaching up to the surface of the ground. The casing is withdrawn, thus allowing the underground material to cave into the open space outside the strainer. Finally, the suction pipe is hung into the strainer. The cross-section of the suction pipe is equal to or smaller than half the cross-section of the strainer, and its lower end is located about 20 in. above the lower end of the strainer. The open space between strainer and suction pipe serves for observing the fluctuations of the water level.

If there is danger that the openings of the wire mesh of the strainer may silt up, as at Lynn, so that the finer particles of the soil may gradually fill the interior of the well, the space between strainer and casing is filled with fine sand prior to pulling the casing.

From Job and Office

Hints That Cut Cost and Time for the Contractor and Engineer

Test Method for Snow Weights

By R. R. Ryan
Newberry, Michigan

THE FOLLOWING test was made for snow weight in the vicinity of Sault Ste. Marie, Mich., at a time when the snow fall and snow melting conditions were nearly ideal for determining the maximum snow load. The equipment used was a carpenter's square, fire-shovel, hand-sleigh, wash boiler and yardstick. Locating a level area free from effects of wind, trees, fences and buildings, 100 measurements at about equal intervals along the circumference of a circle were made by forcing the yardstick vertically through the snow. The average of these 100 readings was about 22 in. Approximately in the center of the area over which the measurements were made, a vertical snow prism of one sq.ft. horizontal section, squared three ways, was cut. The cut-faces of the prism showed light fleecy snow a few hours old on top, grading down to soft snow next to the ground. The prism was lowered into the wash boiler and taken to a grocer's scale, where it weighed 32 lb. net.

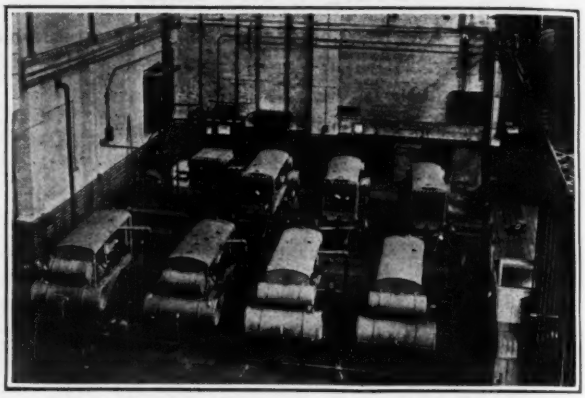
It is for this 32 lb. minimum snow load per sq.ft. of horizontal surface that roofs should be designed in this north country. An ice-rink at Newberry, Mich., recently collapsed under a load of not over 6 lb. per sq.ft.

Compressors Perform Emergency Shop Service During Flood

By Raymond Beechwood
Claremont, N. H.

DURING the period of repair to steam, hot water and compressed air pipes leading from the power house to the main plant of the Sullivan Machinery Company, on Sugar River, a tributary of the Connecticut, repairs made necessary due to the destruction of the bridge carrying service pipes during the early November flood, a battery of portable compressors performed emergency shop service. The company had just completed the construction of a new concrete dam to replace an old timber structure at the upper end of its holdings. On Nov. 3 water was turned into the new penstock and on Nov. 4 the maximum flood height was reached with water pouring to a depth of 5½ ft. above the overfall dam. The stream undermined the foundations of an old suspension footbridge just above the bridge which carried steam, hot water and compressed air pipes from the power house to the main plant. The bridge fell into the stream and was washed away and its cables on the south side rested on the pipe lines referred to which are carried on an overhead trestle to that point. Before the lines could be supported and the weight of the bridge released, the pipes broke and shut off the various services to the main buildings. The machine shop, assembling department and testing department are on the south side of the river and the foundry, pattern shop, warehouses and power house on the north bank, so repair of these lines was immediately necessary.

Emergency service was supplied by seven 4-cylinder 220-ft. units driven by direct connection from 4-cylinder gasoline engines. The receiver of each compressor was hooked up to the main air supply line and all seven ran steadily for three days supplying 1,500 cu.ft. of free air per minute against 85 lb. working pressure. No trouble was experienced in starting up any of the units during the three-day run and only one man was required to watch the operation of these machines during the entire time. Air power was supplied to foundry equipment, sand-blast blowers and to the testing floor for rock drills, for air hoists and for other compressed air equipment throughout the plant.

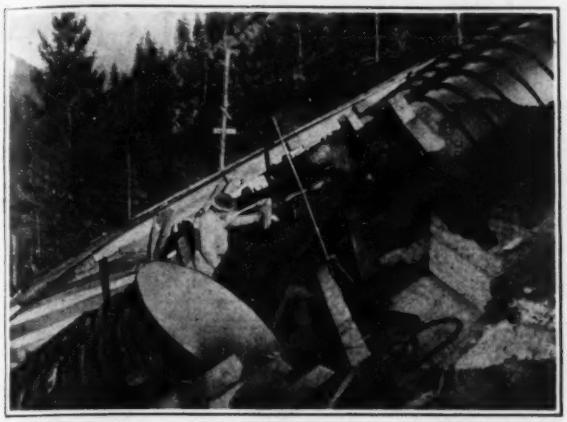


BATTERY OF SEVEN COMPRESSORS SERVING AS EMERGENCY PLANT

Cover for Pipe End on Steep Slope Protects Workmen Inside

THE PENSTOCK for the Bucks Creek hydroelectric plant of the Feather River Power Co. in California is built on such a steep slope that while the line was under construction workmen on the inside would have been in danger if tools or other objects had been inadvertently allowed to slide down the inside of the pipe. As a protection against dangers of this sort light wooden diaphragms were placed inside the penstock near the upper end of a section in which men were at work. By making these of a semi-elliptical shape and fastening them, with ropes, in a more or less horizontal plane, a sort of shelf was provided which would arrest sliding objects and still would not completely close off the draft up the pipe line. This draft was found to be highly desirable as a means of carrying away the smoke and dust resulting from riveting guns working on the inside and also, in the summertime, to keep the air inside down to a comfortable temperature.

At points where expansion joints were to be installed later and hence where uncompleted gaps were left between adjoining sections of the line, a heavy outside covering of boards made circular in shape was used. This could be left tilted at a slight angle so as not to close off the draft up the penstock it protected. Its chief



WOOD COVER FOR PENSTOCK IN WHICH MEN ARE AT WORK

function was to keep out of the pipe tools or other objects that might otherwise have entered with the considerable velocity gained in sliding down the adjoining portion of the line just above.

Constructing 100-Ft. Standard for Checking Steel Tapes

By James E. Earl
City Engineer, Passaic, N. J.

IT IS quite often necessary to check tapes that are repeatedly broken and repaired, against a 100-ft. standard. At times it is impracticable to check such tapes against a standardized tape, so with this in mind we recently completed the following standard measurement.

In the northeast section of the City Hall Park, Passaic, we dug two holes 100 ft. apart, 5 ft. deep and 2 ft. in diameter. These holes were filled with concrete and pieces of 6-in. steel pipe threaded on top to receive a cap were placed about 2 ft. in the soft concrete. Four reinforcing rods of about 1-in. diameter were placed in the pipes and down through the soft concrete. The pipes were then filled with concrete flush with the top and brass rods 1 in. square and 1 ft. long were placed in the concrete in the pipe and left sticking about ⅛ in. above the top of the steel pipe. The brass bar at the north or zero end had a crosscut made with a fine blade saw on the top, and the brass bar at the southerly or 100-ft. end had a line cut on it parallel to the tape only. These brass bars were set so that the southerly end bar was exactly 1½ ft. below the northerly end. The pipe caps were then screwed on and the whole thing left to set.

We then purchased a standard 100-ft. tape and sent it to the Bureau of Standards to be tested and numbered. This tape was similar to an ordinary surveyor's 100-ft. tape except that instead of the 100-ft. mark, it was graduated in degrees Fahrenheit. By placing a thermometer alongside the tape and reading the corresponding temperature graduation, the exact location of the 100-ft. mark was obtained.

About a year and a half elapsed and then levels were taken on the two brass plugs. Frost had not affected them, so we proceeded to put the 100-ft. mark on the southerly monument as follows. Stakes were driven in the ground every 5 ft. between the two monuments and a strip of wood 1 in. by 2 in. was fastened to these stakes in such a manner that this strip was level with the top of the brass plug at the northerly monument. This strip passed directly over the southerly monument 1½ ft. above it. Directly over this monument the strip of wood had a recess cut in it 1 in. square. Posts were driven in the ground, a hook and turnbuckles attached at the zero end and a hook, tension scale and turnbuckle attached at the 100-ft. end of the tape. The tape was then laid on this strip. By means of the turnbuckle at the zero end, the zero of the tape was made to coincide with the crosscut on the brass plug. In the same manner, the turnbuckle at the 100-ft. end of the tape was tightened until the tension handle registered 10½ lb., the tension at which it had been standardized. The tape was now level and supported throughout at its standard tension and the 100-ft. mark came over the 1-in. recess cut in the wooden strip without anyone touching the tape. Thus personal errors were eliminated.

The temperature in the morning was about 60 deg., in the afternoon about 70. About 10 o'clock in the morning the tape was stretched along this strip and left that way for about an hour. Two thermometers were laid on the bench, one at the zero end and one at the 100-ft. end. They both read 64 deg. so at 64 deg. on the tape a plumb-bob was let down through the recess, cut in the strip and a pencil mark was made on the brass plug. In the afternoon the tape was again laid out and the thermometer placed on the wooden bench. The thermometer read 72 deg., and plumbing down from that temperature graduation on the tape, we found the point of the plumb-bob exactly on the pencil mark made in the morning. Each one of four persons made this same measurement and when it was found they were identical, the mark was cut in the brass plug with a fine blade saw.

Waterproofing a Cellar

By F. E. Barnes
Building Valuation Engineer, New York Central Lines, New York City

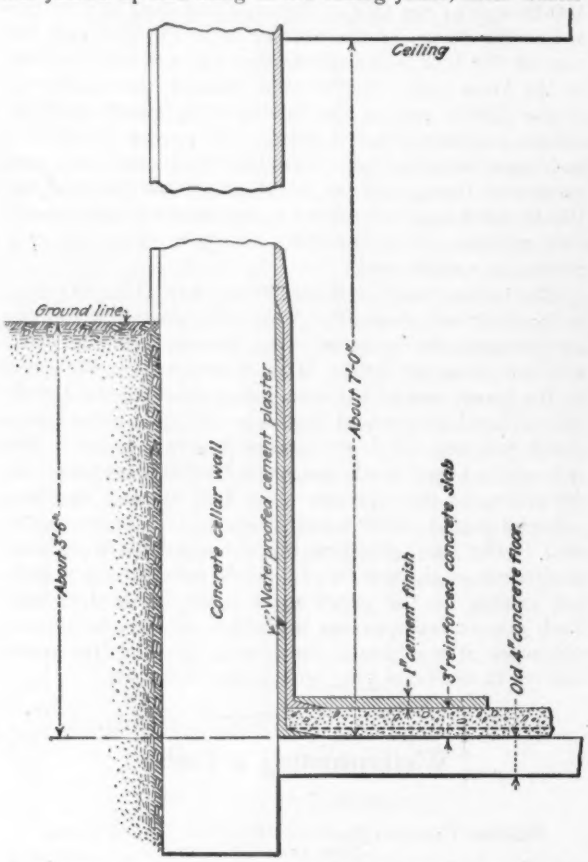
RECENT experience in waterproofing a cellar has proved so successful that a record of the procedure is worth while. The cellar in question, during exceptionally heavy rains, would have from 6 in. to 3 ft. of water over the floor. The ground was very porous, and there was an upward pressure on the floor caused by a 3½-ft. head of water which had cracked the floor almost all the way around the cellar at the wall line, as well as in various other places.

First, removing everything from the cellar, including the partitions, the inside of the cellar wall was given a coating of waterproof cement plaster from the ground line to the floor, and extending about 3 or 4 in. out over the floor. A 3-in. reinforced-concrete slab, using ⅜-in. reinforcing bars in both directions spaced about 18 in. apart, was next laid, following which a second coat of waterproof cement plaster was placed over the entire slab. On top of this, a waterproof cement finish known as Hydratite and manufactured by the A. C. Horn Co., Long Island City, was placed covering the entire floor.

In order to take care of the upward pressure of water that occurred during heavy storms, the old floor was thoroughly cleaned and covered with a cement grout, so that the new floor slab would bond with it as well as possible. Weight was added to the slab by placing the furnace upon it and by making the partitions fit tightly,

the studding being cut to such length to permit of a driving fit, thus transferring a considerable load from the first floor to the cellar floor.

The furnace could not be raised and it was therefore necessary to keep the new floor at that point at the same elevation as the old floor. In order to do this, the furnace was jacked up and hung to the ceiling joists, after which



METHOD OF WATERPROOFING OLD CELLAR FLOOR SLAB AND WALL

the base was removed and the old floor beneath sledged out. The new floor was then constructed underneath the furnace at the same elevation as the old one had been.

The size of the cellar was 24x25 ft. with a 7-ft. clear headroom. The cost of doing the work was approximately $375, the labor cost being $175 and the material $200. Since the work has been completed there have been two storms fully as bad as any experienced previously. No leaks have developed.

Simple Design for Riveted Steel Penstock Wye

By Oswald Speir, Jr.
Designing Engineer, Constant Angle Arch Dam Company, San Francisco, Calif.

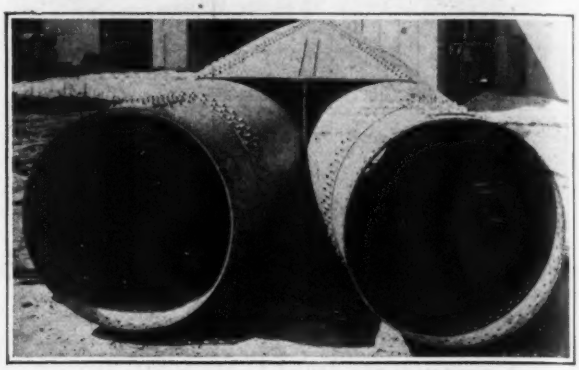
A DESIGN for a heavy riveted steel penstock wye has been developed for the Bucks Creek project of the Feather River Power Co. in California involving comparatively simple construction and yet having great strength. The design was made easy of fabrication by eliminating difficult riveting throughout, particularly in the crotch. The plates are all stressed in tension and

From Job and Office

Hints That Cut Cost and Time

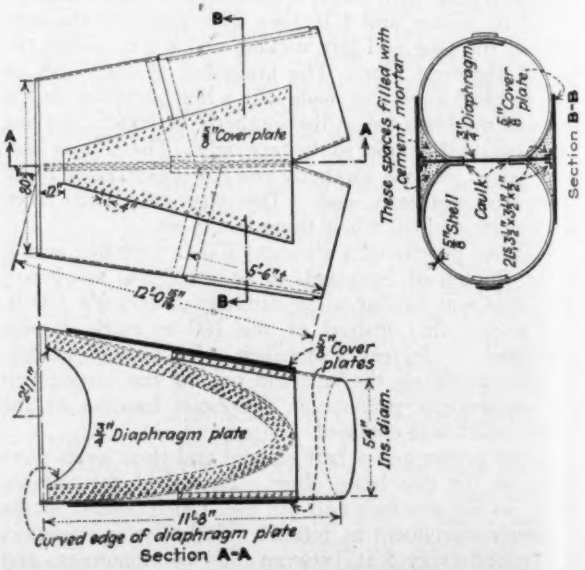
the rivets in shear without tension or bending, thereby overcoming one of the usual defects of wye construction. The method is applicable to practically any riveted wye of this type.

The wye divides a pipe of 80-in. diameter into two 54-in. laterals. The bottom of the pipe is level, the top



RIVETED STEEL WYE FOR BUCKS CREEK PENSTOCK

sloping from 80 in. at the up-stream end to the 54-in. diameter at the ends of the two legs. The legs are developed as truncated cones and at their plane of intersection a ¾-in. diaphragm plate is inserted to which the legs are riveted. The diaphragm extends from the plane of the top to the plane of the bottom and from the crotch to the large end of the wye as indicated in the drawing. Cover plates are riveted to the top and bottom, being connected to the diaphragm by angles. The triangular spaces between pipes, cover plates and diaphragm are filled with a cement mortar put in when the wye was erected. This cement mortar was placed with the wye in a vertical position so that when the mortar was poured in from the top gravity would insure filling the space completely.



DETAILS OF A SIMPLE PENSTOCK WYE DESIGN

From Job and Office

For Contractor and Engineer

The function of the diaphragm plate is to take care of the unbalanced load at the center in direct tension and also to eliminate difficult riveting in the crotch. The cover plate transfers hoop tension from one leg to the other without stressing the curved connection with the diaphragm plate. By filling the open triangular space with mortar the inside row of rivets in the crotch is relieved from bending stress due to the water pressure in this area. Each leg was fabricated separately as was the diaphragm and cover plate with connection angles. The three parts were then brought together, bolted up and riveted.

Fabrication was by the Western Pipe & Steel Company. Engineering work on the project was done for the Feather River Power Co. by the Constant Angle Arch Dam Co. of San Francisco.

Wall Castings for Cast-Iron Pipe

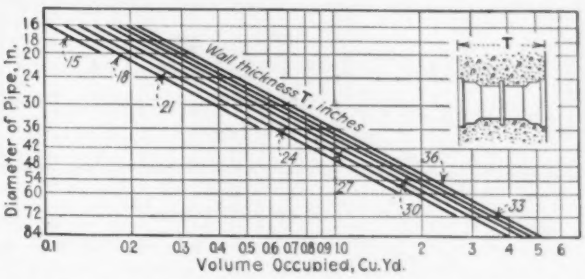
By R. T. Regester
Junior Civil Engineer, Bureau of Sewers, Baltimore, Md.

SPECIAL wall castings are used by various engineers wherever cast-iron pipes pass through the walls of sewage-works and water-works structures as a means of preventing leakage around the pipes and breaking caused by settlement. Lead joints having some flexibility are provided at the points most needed while a center flange embedded in the wall offers resistance to the flow of water along the outside surface of the pipe.

As data on wall castings are lacking, the accompanying tables and diagram may be found useful for the laying-out and estimating of pipe lines involving these castings. In Table I, with the exception of the center flange, dimensions conform with the Standard Specifications for Cast Iron Pipe and Special Castings of the American Water Works Association. All bells are Class "B" for special castings and the thickness of pipe barrel is the same as Class "B" pipe. The minimum wall thickness for which a casting can be used is given together with the corresponding weight of casting for this thickness of wall.

Table II gives the dimensions of wall castings having a flange on one end for use in wet wells or tanks where a bell mouth connection is needed. End flanges conform with the American 1914 Standard. For the flanged joint ordinary bolts can be used by placing the bolt heads on the inside of the wall; but it is preferable to use stud bolts with the flange holes tapped, thus providing easier replacement of a sheared bolt. The minimum wall thickness given allows a clearance, *L*, for placing ordinary bolts if these should be used.

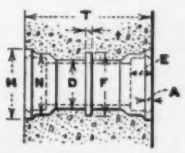
The weight per lineal inch given in Table II may be used to find the weights of wall castings in both tables for wall thicknesses greater than the minimum.



VOLUME OF WALL OCCUPIED BY WALL CASTINGS

TABLE I—DIMENSIONS OF WALL CASTINGS WITH TWO BELLS

All dimensions in inches; weights in pounds.

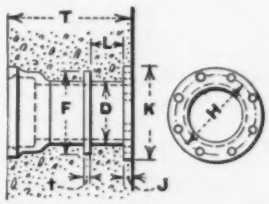


Nominal Diam.	Outside Diam. of Pipe D	Outside Diam. of Bell Face M	Outside Diam. of Bell Body N	Center Diam. F	Flange Thickness t	Depth of Socket E	A	Min. Wall Thickness T	Weight of Casting for Wall T
3	3.96	7.26	5.96	6	0.50	3.5	1.25	11	44
4	5.00	8.30	7.00	7	0.50	4.0	1.50	12	46
6	7.10	10.60	9.20	10	0.75	4.0	1.50	13	72
8	9.05	13.00	11.50	12	0.75	4.0	1.50	13	140
10	11.10	15.10	13.60	14	0.75	4.0	1.50	14	157
12	13.20	17.40	15.80	16	0.75	4.0	1.50	14	184
14	15.30	19.50	17.80	18	0.75	4.0	1.50	14	245
16	17.40	22.00	20.20	20	0.75	4.0	1.75	14	318
18	19.50	24.30	22.40	24	1.00	4.0	1.75	15	408
20	21.60	26.60	24.60	26	1.00	4.0	1.75	15	418
24	25.80	31.00	28.90	30	1.00	4.0	2.00	15	553
30	32.00	37.60	35.30	36	1.00	4.5	2.00	16	728
36	38.30	44.90	42.10	44	1.50	4.5	2.00	18	1,300
42	44.50	51.50	48.50	50	1.50	5.0	2.00	19	1,745
48	50.80	58.40	55.10	56	1.50	5.0	2.00	19	2,183
54	57.10	65.30	61.70	62	1.50	5.5	2.25	21	2,980
60	63.40	71.80	68.20	68	1.50	5.5	2.25	21	3,246
72	76.00	85.65	81.65	82	2.00	5.5	2.25	23	4,952
84	88.54	98.79	94.99	94	2.00	5.5	2.50	25	7,983

All bells are Class-B for special castings.

TABLE II—DIMENSIONS OF WALL CASTINGS WITH BELL AND FLANGE

All dimensions in inches; weights in pounds.



Nominal Diam.	Diam. K	End Flange: Diam. of Bolt Circle H	End Flange: No. of Bolts	End Flange: Diam. of Bolts	End Flange: Thickness J	L	Min. Wall Thickness T	Weight of Casting for Wall T	Weight per Lin.In. Greater than T
3	7.50	6.00	4	¾	0.75	3.5	10	35	1.2
4	9.00	7.50	8	¾	0.94	3.5	11	44	1.7
6	11.00	9.50	8	¾	1.00	4.0	12	69	2.6
8	13.50	11.75	8	¾	1.13	4.0	12	118	3.5
10	16.00	14.25	12	¾	1.19	4.5	13	146	4.9
12	19.00	17.00	12	¾	1.25	4.5	13	184	6.4
14	21.00	18.75	12	1	1.38	5.0	14	243	7.9
16	23.50	21.25	16	1	1.44	5.0	14	307	9.5
18	25.00	22.75	16	1¼	1.56	5.5	15	389	11.5
20	27.50	25.00	20	1¼	1.69	6.0	16	446	13.6
24	32.00	29.50	20	1¼	1.88	6.5	17	610	18.1
30	38.75	36.00	28	1¼	2.13	7.0	18	860	26.1
36	46.00	42.75	32	1¼	2.38	8.0	20	1,421	34.9
42	53.00	49.50	36	1¼	2.63	9.0	22	1,963	45.2
48	59.50	56.00	44	1¼	2.75	9.0	23	2,490	57.3

Dimensions of bells and center flanges same as in Table I.

From the diagram the volume of wall occupied by castings of either type for various wall thicknesses may be readily obtained for estimates in which the practice of deducting all openings is followed. As an example in the use of the diagram, a 48-in. casting in a 30-in. wall will be found to occupy 1.5 cu.yd. of wall volume.

Letters to the Editor

A Forum for Discussion of Views of Engineers and Contractors

Unionization and Engineers

Sir—In your issue of November 3 is a letter from J. P. J. Williams entitled "Unionization for Engineers." In this letter Mr. Williams first commends the American Society of Civil Engineers for its appeal to the Board of Estimate and Apportionment of New York City for better engineering salaries, and then states his belief that all such appeals "are only too likely to be fruitless and abortive." Probably not many engineers can agree with this opinion. For several years the Am.Soc.C.E. has been criticized for not taking more interest in the economic welfare of the engineer, in spite of the fact that the society was not organized for that purpose. It is not very encouraging now to be told that any efforts it may make are likely to be valueless. Fortunately, the result of the request in which the Am. Soc. C. E. cooperated is the inclusion in the 1928 budget of $600,000 for increases in engineering salaries. The writer knows at least one other city where the engineering employees would welcome such a "fruitless appeal" from the Society.

Anyone familiar with the status of the average engineer in this country cannot help but agree with Mr. Williams as to the injustice of present conditions, but few thinking engineers are of the opinion that unionism is the only remedy or the best remedy. Mr. Williams speaks of "the salesman who expects and receives a minimum of $5,000 annually, with more usual returns of $10,000 to $20,000." Is this the result of a union with a schedule of minimum salaries, etc.?

As a matter of fact, in spite of the large number of subordinate positions, engineering very much more nearly resembles the professions of law and medicine than it does the trades such as carpentering, plastering and bricklaying. Therefore, the logical solution of these economic problems would seem to be the banding together of all engineers in one common organization, such as the American Medical Association or the bar association. Such an organization of engineers already exists and has accomplished much. The only reason that it has not accomplished more is because it has not been given the support to which it is entitled. If 75 per cent of the engineers of the United States were active members of the American Association of Engineers, the results it could achieve in two or three years would be almost unbelievable. But so long as the "spirit of individualism" is so pronounced among engineers that the majority of them will not give five cents a day to support such an organization as the A.A.E., its achievements are bound to be limited.

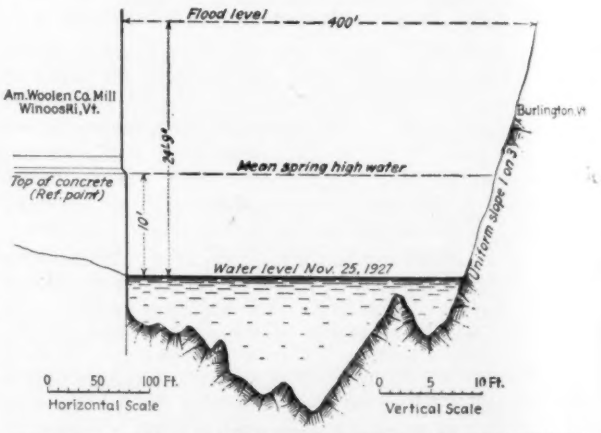
In addition to this "spirit of individualism," the idea of joining a labor union or affiliating with the American Federation of Labor is distasteful to most thinking engineers, as it should be, so that effective "unionization of engineer-workers" does not appear to be very feasible. What little sentiment favorable to unionization has been built up, has been due largely to the false idea among some engineers that all they would need to do would be to join a union and pay dues for a year or two and their salaries would be increased for them. This same false impression seemed to exist a few years ago in regard to the American Association of Engineers. The result of it was that the association was attacked from some quarters as a union, and on the other hand, many who joined, dropped out after a year or two because their salaries were not increased by their merely paying dues to the A.A.E. In most cases, they not only failed to contribute one constructive idea to the cause, but they found fault with everything that was done or suggested by anyone else. In short they had little or no conception of the meaning of cooperation, and without that, no organization can hope to gain permanent success.

Of course larger numbers of engineers may be driven into unionism, in spite of their distaste for it, if they continue to be given the impression that this is the only way in which their appeals can receive any recognition, as apparently was done at Chicago. If the powers that be, whether political, industrial or commercial, are short-sighted enough to continue to permit this impression to become widespread, it will be a big help to the American Federation of Labor in its efforts to unionize engineers. If the technical societies continue to close their eyes to this situation, either for so-called ethical reasons or because they are told their efforts will probably be fruitless and abortive, that too should increase the sentiment for unionization.

Los Angeles, Calif. W. C. HOGOBOOM.
Dec. 5, 1927.

Flood Discharge Estimated for Winooski River

Sir—As an indication of the flood discharge during the early November flood, I am enclosing a cross-section of the Winooski River at Burlington, Vt. The sketch herewith is taken on a line from the downstream corner of the upper mill of the American Woolen Company in Winooski to a point in Burlington directly opposite. In general it follows the line of our pontoon bridge. The principal volume of the flood water undoubtedly passed between this mill and the top of the first rise on the Burlington side. If the estimates of the observers that the mean velocity was 20 ft. per second were correct it would appear that a volume of 100,000 sec.-ft. as was also estimated was decidedly in error. The cross-section is at least 400 by 25 with an area of 10,000 sq.ft. A mean



CROSS-SECTION OF WINOOSKI RIVER AT BURLINGTON

velocity of 20 ft. per second would give 200,000 sec.-ft. as the total discharge.

A slight rise in the water level due to general rains on Nov. 17 caused a rise of approximately 3 ft. above the water level on Nov. 25 as shown in the sketch. The velocity with the 3-ft. rise was 13 ft. per second in the center of the stream. On Nov. 25 the velocity was 3 ft. per second also in the center of the stream. On Nov. 16 the water stood 1 ft. below this level and the velocity was not over 1 ft. per second in the center of the stream. It was very noticeable that as the water level rose not only did the velocity in the center of the stream increase rapidly but also the width of the high velocity stream expanded enormously. From the result of this slight rise I would not hesitate to believe the 20 ft. per second mean velocity although prior to it I was extremely skeptical. My readings of the 13-ft. per second velocity were on surface floats but a sufficiently large number of readings under accurate conditions of measurement were made to insure the result. I borrowed a current meter from the university but was unable to keep it under water as there was no rod with it and we had to depend on the lead which proved too light to hold the meter under the surface.

Fort Ethan Allen, Vt., L. R. GROVES, JR.,
Nov. 29, 1927. 1st Lieut., Corps of Engineers,
U. S. Army.

News of the Week

Current Events in the Civil Engineering and Contracting Fields

Estimate Board Holds Hearing on Additional Water Supply for N. Y.

A public hearing on the Delaware River and Rondout Creek project for an additional water supply for New York City was held by the Board of Estimate and Apportionment, New York City, on Dec. 15 and then adjourned for a week. Notice of court action to protect the rights of the State of New Jersey and the City of Trenton, N. J., was given officially. Similar action in behalf of Pennsylvania is considered probable. No action was taken on the proposal of the Titus Water Engineering Corporation to install wells and pumps on Long Island to provide an "emergency" supply of 150 m.g.d., which has been before the board for more than a year.

The project on which the hearing was held contemplates a supply of 600 m.g.d. from five New York tributaries of the Delaware River and 70 to 100 m.g.d. from Rondout Creek, a tributary of the Hudson, together with the diversion into the existing Catskill aqueduct system of water from some of the higher levels of the Croton system. This project was submitted to the Board of Estimate by the Board of Water Supply (Thaddeus Merriman, chief engineer), early this year (see *Engineering News-Record*, Aug. 11, 1927, p. 218) and referred to Arthur S. Tuttle, chief engineer Board of Estimate for report. Mr. Tuttle's report, dated Oct. 6, 1927, endorsed the Delaware and Rondout project and recommended that various necessary steps be taken to meet legal requirements, including a call for the hearing of Dec. 15, and also that the Board of Water Supply proceed with plans for the project but without commitments as to property or contracts until the corporation counsel has given assurance that the city has the right to make the proposed diversion.

Besides endorsing the Delaware River project Mr. Tuttle recommended that the Department of Water Supply, Gas and Electricity consider what changes in water rates (*not increased since 1857*) should be made to prevent a deficit; consider what should be done to reduce per capita water consumption during the building of additional works; make plans at once (a) to utilize the Croton supply to its full capacity; (b) restore existing driven well stations on Long Island to their full capacity for emergency use, and (c) that it should explore the Long Island deep water resources to the end that they may be availed of to their utmost capacity during the period which must elapse after the Croton supply has been fully developed and up to the time when the new Delaware supply can be introduced."

As to putting down more wells on Long Island, without mentioning the 150-m.g.d. Titus well project, referred to above, Mr. Tuttle says "there seems reason to believe that "an additional 30 m.g.d. can be obtained from Long Island sources by an extension of the deep well system." (For synopsis of the Titus proposal, as endorsed by Nicholas J. Hayes, Jr., commissioner of Water Supply, see *Engineering News-Record*, Dec. 9, 1926, p. 672.)

U. S. Chamber Enunciates Flood Control Program

As the result of a referendum vote just concluded among its membership, the Chamber of Commerce of the United States will recommend to Congress the following Mississippi flood-control program: (1) The federal government should hereafter pay the entire cost of constructing and maintaining works necessary to control floods of the lower Mississippi River; (2) the federal government should assume sole responsibility for locating, constructing and maintaining such works; (3) there should be adequate appropriation to insure efficient, continuous and economic work, the funds to be made available as needed; (4) flood control of the Mississippi should be dealt with in legislation and administration on its own merits, separate and distinct from any other undertaking.

The vote was taken among the Chamber's 1,500 member organizations on the report of a special committee of outstanding business men and engineers which early in the fall completed its study of the subject with a trip through the flooded areas.

Highway Engineers Form Consulting Company

Consulting highway engineering service is offered through the recent organization of the Highway Engineering Bureau and the opening of offices in the National Press Building in Washington, D. C. The organization will offer consulting work in highways, transportation and associated industries. Advisory engineers include Charles M. Upham, president; Fred E. Schnepfe, vice-president; A. T. Goldbeck, Prevost Hubbard, Maurice Holland, Fred A. Reimer, Henry G. Shirley, Earl Stafford, Wm. A. Van Duzer, Dr. J. A. L. Waddell and Shortridge Hardesty. The services of the bureau will be particularly available to counties, states, cities, foreign countries, bonding companies, contractors, banks and civic associations.

Propose Demonstration Settlement of Southern Reclamation Projects

Progress in the study of Southern land reclamation and settlement possibilities was discussed at a conference held in the auditorium of the Interior Department, Washington, D. C., on Dec. 14 and 15, under the auspices of the Bureau of Reclamation. The theme of the conference was the investigation that has been carried out by the Bureau of Reclamation during the past year to develop a plan for reclamation and organized settlement of a number of demonstration tracts. Tracts in the Carolinas, Georgia, Florida, Alabama, Mississippi and Tennessee were selected by the authorities of the respective states, and were then examined by special boards of the Bureau of Reclamation as to soil, land ownership, suitability for group settlement, nearness to markets, and the like. Clearing and drainage and similar improvement work was also studied. These investigations were made by direction of Congress, $15,000 having been appropriated in each of the past two years for the purpose.

Detail reports of the results of the studies were given at the conference in papers by Dr. Elwood Mead, commissioner of the Bureau of Reclamation, and George B. Kreutzer, director of reclamation economics. A tract of 24,000 acres near Albany, Ga.; a 27,000-acre tract at Mayland, Tenn.; a 9,000-acre tract at Richton, Miss.; and an 8,000-acre tract near Charleston, S. C., were found particularly favorable for development. Each would be divided into tracts of 80 to 120 acres. Clearing and other reclamation and the development of farms into going properties would make the total cost per farm $10,000 to $12,000. The settlement plans that are in view include expert planning and supervision, provision of money to be advanced to settlers for improvement and equipment of farms, and co-operation of the local people with state and federal agencies to institute centralized marketing and the like.

Papers by Dr. W. W. Long, South Carolina; Senator L. B. Tyson, Tennessee; Dr. E. C. Branson, North Carolina; C. S. Ucker, Georgia; Hugh MacRae, North Carolina; Congressman W. M. Withington, Mississippi; Daniel C. Roper, Georgia; J. M. Patterson, Georgia; W. F. Lineberger, North Carolina; and L. J. Folse, Mississippi, emphasized the need of the South for fuller agricultural development. Many millions of acres of abandoned land are to be found in

that part of the country, and much land that is nominally farmed is in a condition of very low yield, it was brought out.

Dr. Mead proposed that a foundation fund of several million dollars be created by private contributions for the initial expense of development, to be reimbursed by the settlers on a deferred payment plan, and that this fund be supplemented by federal and state appropriations to cover the supervisional expense. A different view was taken by the conference, however, which at the close of the session passed a resolution requesting Congress to make an appropriation of $14,000,000, of which $2,000,000 is to be allotted to a project in each of the states of Alabama, Florida, Georgia, Mississippi, North Carolina, South Carolina and Tennessee, the money to be expended under the direction of the Bureau of Reclamation for land improvement and colonization on the model tract. The resolution provides that the appropriation be surrounded by safeguards which "will insure reasonably that the entire fund appropriated will be returned to the treasury of the United States."

Welded Roof Truss for General Electric Company Tested at Trenton, N. J.

On Dec. 16 a proof test of a welded roof truss was carried out at the Trenton, N. J., plant of the American Bridge Co. for the General Electric Co., of Schenectady. The truss was 58 ft. long center to center of supports and 7 ft. deep between chord centers, and had webbing of Pratt type arranged in six panels. The chords each consisted of an 8-in. H-beam with web horizontal, the diagonals were two channels with flanges out, laid against the vertical outer faces of the chord flanges, while the posts were 7-in. I-beams. The welding at the joints is illustrated in one of the accompanying views.

The load for which the truss is designed amounts to 79,000 lb. A test load of 116,000 lb. was applied by I-beams laid on a loading frame resting on the test truss and steadied by two similar trusses placed so as to form a braced two-bay section of framing set up under the yard crane of the bridge shop. Thus, a test load of about 50 per cent in excess of the working load was applied, representing service under about twice the intended live-load of the truss. None of the members or joints showed any distress in the tests. Deflections were measured at each stage of loading, and reached a maximum of about $\frac{3}{4}$ in.

The connections of this truss were made by arc welding throughout. The weld fillets were $\frac{5}{16}$ to $\frac{3}{8}$-in. in size and were made with $\frac{5}{16}$-in. bare welding wire supplied with current from a single-operator motor-generator set. The joints were designed for a shear stress of 3,000 lb. per lineal inch of fillet, while the members of the truss in general were designed on a 16,000-lb. base stress.

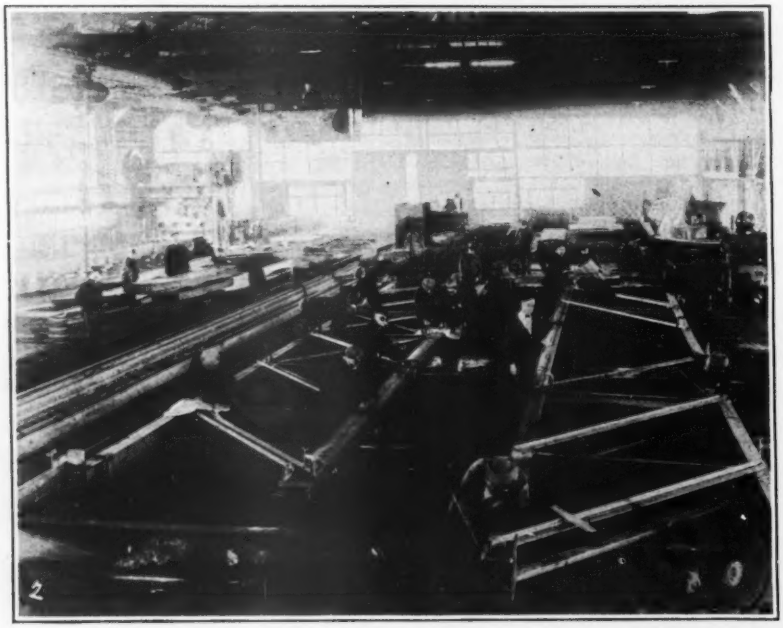


FIG. 1—ROOF TRUSSES IN WELDING SHOP AT TRENTON

General Patrick Is Retired

Major-General Mason M. Patrick, chief of the Army Air Corps, having reached the statutory age of 64, has been retired.

General Patrick is a native of West Virginia and was an honor graduate at the U. S. Military Academy in 1886. Upon his graduation he was assigned to the Corps of Engineers and served continuously in that arm until he was made chief of the Air Service in 1921. Among his earlier important assignments, he was a member of the Board of Officers which supervised the raising of the battleship Maine, served as commandant of the Engineers School and was chief engineer of the American forces in Cuba in 1906-08.

At the commencement of the World War, General Patrick was assigned as chief engineer of the Lines of Communication and afterwards became Director of Construction and Forestry. On May 29, 1918, he was designated by General Pershing as chief of the Air Service of the A.E.F.

His services in that capacity brought him wide distinction in all the allied armies. He was appointed chief of the Air Service of the United States Army with the rank of Major General on Nov. 5, 1921.

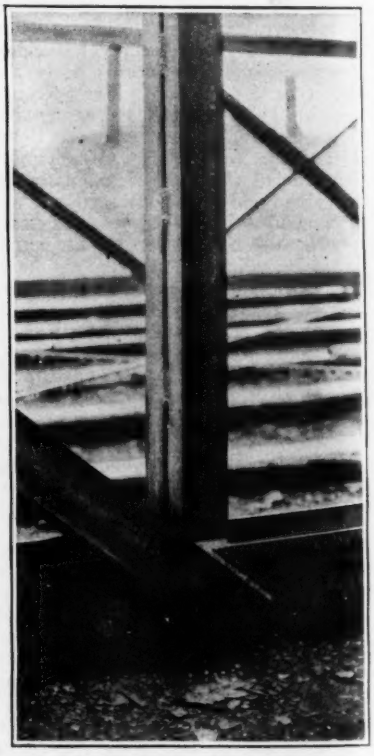


FIG. 2—BOTTOM-CHORD JOINT OF WELDED ROOF TRUSS WITH H-BEAM CHORDS

Irrigation System Advised for Central Park

An irrigation system at a cost of $265,360 is the chief item in the $873,920 estimate for the cost of rehabilitating Central Park, New York, recommended to Walter R. Herrick, park commissioner, by Hermann W. Merkel, general superintendent of the Westchester County Park Commission, White Plains, N. Y. Many miles of fences, to protect plantings, are advised, at a cost of $40,000.

New York Governor Orders Inquiry Based on Queens Sewer Charges

Evidence brought out in hearings before the board of assessors and the commissioner of accounts of New York City on alleged negligence, incompetence and fraud in connection with sewer construction in Queens borough, has been made the basis for charges presented to Governor Smith, looking to the removal of borough president Maurice E. Connolly. The governor without delay appointed Supreme Court Justice Townsend Scudder to conduct an investigation. Justice Scudder states that he will serve the charges on President Connolly and start hearings about the first of the year.

The other hearings have in the meantime been suspended at the instance of Mayor Walker until Justice Scudder begins his action. Incidentally, Mayor Walker made plain that the hearings before the Commissioner of Accounts were only incidentally concerned with Queens but had been started to determine whether a more efficient way cannot be devised to carry on city business than the present autonomous borough plan. Thus this hearing will probably involve the sewer and highway departments of all boroughs of New York City.

The charges presented to Governor Smith were as follows:

1. Gross waste of public funds.
2. Unlawful preparation and acceptance of specifications in contracts for sewer construction.
3. Fraud on taxpayers by disregard of engineer's estimates.
4. Employment of incompetent and unqualified borough engineers to handle contracts involving millions of dollars of public funds.

At the board of assessors hearing Dec. 15, President Connolly made his first formal denial of charges in the form of a letter prepared for him by Prof. William H. Burr, consulting engineer. Professor Burr's letter stated that the sewers in Queens were not more expensive than necessary to meet anticipated conditions, that specifying 1:1:2 concrete was reasonable and proper and that other provisions of the specifications were justifiable especially since they could be altered in accordance with the engineer's judgment. The letter further stated that the specifications did not preclude the use of other precast concrete pipe than the lock joint design.

Vermont Arranges Finance Plan for Reconstruction

Through the floating of a $8,500,000 bond issue and the establishment of a credit corporation to assist farmers, the state of Vermont has practically completed finance plans for reconstruction. The bond issue is the first that has been floated in the state since 1919 when an issue of $1,500,000 soldiers' bonus was marketed. The present issue will be due in from five to thirty years and the proceeds will be used to reconstruct roads and bridges.

The Committee on Rehabilitation Credits of the New England Council has evolved a plan for aiding the farmers through the creation of the Vermont Credit Corporation with a capital of $1,000,000. A second corporation, the New England Flood Credit Corporation, has been formed to sell the stock of the first.

Moffat Tunnel Completed

On Dec. 10 the last excavation was made and the last of the 10-ton steel sets for the lining was placed in the Moffat Tunnel. The only work remaining was 4,200 cu.yd. of lining to be placed by the concrete guns, and then the laying of the track, which will have 110-lb. rails. It is expected that the first train will be run in January.

Boston Engineers Hear Twelve Papers on Welding

About 300 members of Affiliated Technical Societies of Boston and their guests were present at the morning, afternoon and evening sessions of the welding meeting held Dec. 14 by that organization. The morning session was devoted principally to structural welding, with an introductory paper by F. M. Farmer, president, American Welding Society, New York, on "General Principles of the Various Welding Processes." This was followed by papers by Gilbert D. Fish, consulting engineer, Westinghouse Electric & Manufacturing Company, New York, and by Andrew Vogel, General Electric Company.

The luncheon and the afternoon session were held jointly with the New England Water Works Association, one of the member organizations of The Affiliated Technical Societies of Boston. The program of the afternoon session, which was devoted principally to pipe welding, included papers read by:

Le Roy Edwards, Industrial Engineering Department, Air Reduction Sales Company, New York; D. H. Deyoe, Industrial Engineering Department, General Electric Co., Schenectady, N. Y.; Robert L. Browne, district sales manager, Metal & Thermit Corporation, Boston, Mass.; Dr. A. Krebs, treasurer, General Welding & Equipment Co., Boston; and C. A. McCune, director of research, American Chain Company.

At 6:30 p.m. the informal dinner and smoker was held, at which Col. Lewis E. Moore, chairman of the Affiliated Societies of Boston presided. The evening session, at The Boston City Club, included the following speakers:

Fred T. Llewellyn, representing the American Society of Civil Engineers on the Joint Structural Welding Committee; W. T. Ober, Thomson Electric Welding Company, Lynn, Mass.; C. W. Babcock, sales engineer, Westinghouse Electric and Manufacturing Company, Boston; and P. Alexander, Thomson Research Laboratory, General Electric Company, West Lynn, Mass.

Fifty Years of Engineering Reviewed At Sessions of Philadelphia Engineers

Last week in Philadelphia men prominent in all branches of the engineering profession reviewed the progress of the past fifty years in engineering, interpreted present tendencies and looked ahead to greater engineering achievements in a series of gatherings marking the golden anniversary of the Philadelphia Engineers Club. Leading men from all parts of the country and representatives of all engineering and scientific branches were guests of the club at the sessions held Dec. 16 and 17 at the Bellevue-Stratford Hotel.

Engineering of the past was pictured at the first session by Willard T. Chevalier, general manager of *Engineering News-Record* and *Construction Methods*, both McGraw-Hill publications, and by Ambrose Swasey, past-president of the American Society of Mechanical Engineers. These addresses were followed by a buffet supper, entertainment and smoker.

On Saturday morning Josiah Penniman, provost of the University of Pennsylvania, conferred the honorary Doctor of Laws degree upon Howard Elliott, chairman of the board of the Northern Pacific Railway; John Hays Hammond, mining engineer, and Charles M. Schwab, president of the American Iron and Steel Institute.

At the luncheon session on Saturday, Dr. Schwab, Dr. Elliott and Frank B. Jewett, vice-president of the American Telephone & Telegraph Company, appraised the engineering achievements of today. In the evening at a formal dinner attended by 400 members and guests, J. H. M. Andrews, president of the club, conferred honorary life membership on Charles Billin, who was a prime mover in organizing the club and has been its secretary since its inception. This was followed by addresses by Samuel Rea, honorary member of the club and former president of the Pennsylvania Railroad, and Dean Dexter S. Kimball of Cornell University. The tenor of these two speeches was a forecast as to what engineering might achieve by 1977.

Eight Lake Cities Must Prepare Sewage-Treatment Plans

Eight incorporated places on Lake Michigan have been ordered by the Indiana State Board of Health to make sewage-treatment plans for preventing the pollution of public water supplies drawn from the lake and to file the plans with the board for approval. Formal reports as to the action taken must be filed with the board by June 1, 1928. The cities notified are Hammond, Whiting, East Chicago, Gary, Michigan City, Hobart, Highland and Munster. The official notice, dated Dec. 10, was signed by William F. King, secretary Indiana Board of Health, Indianapolis.

State Department Designates U. S. Members on Permanent International Road Congress

Upon recommendation of the U. S. Bureau of Public Roads, Secretary of State Frank B. Kellogg has named 15 men representatives of the United States on the Permanent Commission of the International Association of Road Congresses. A number of prominent engineers are in the list, which is as follows:

W. J. Carr, Assistant Secretary of State; Charles E. Herring, commercial attache, Department of Commerce; H. H. Kelly, trade commissioner, Department of Commerce, Paris; Thomas H. MacDonald, chief, U. S. Bureau of Public Roads; Frank T. Sheets, president, American Association of State Highway Officials; C. M. Babcock, president, American Road Builders Association; John N. Mackall, chairman, Maryland State Roads Commission; Dr. L. S. Rowe, director-general, Pan-American Union; Roy D. Chapin, president, National Automobile Chamber of Commerce; Pyke Johnson, National Automobile Chamber of Commerce; Dean F. L. Bishop, University of Pittsburgh; Col. A. B. Barber, U. S. Chamber of Commerce; E. J. Mehren, vice-president, McGraw-Hill Publishing Co., Inc.; Stephen G. Porter, chairman, House Committee on Foreign Affairs; William E. Borah, chairman, Senate Committee on Foreign Relations.

Bill Would Establish Hydraulic Laboratory in Standards Bureau

A bill has been introduced into the Senate by Senator Ransdell authorizing the establishment of a national hydraulic laboratory in the Bureau of Standards. The bill was drafted by John R. Freeman. The bill proposes a national hydraulic laboratory "for the determination of fundamental data useful in hydraulic research and engineering including laboratory research relating to the behavior and control of river and harbor waters, the study of hydraulic structures and water flow, the development and testing of hydraulic instruments and accessories, and including such co-operation with other government departments, the states, technical and scientific institutions and societies as may be deemed necessary."

City Supervisor To Be Appointed at Englewood, N. J.

A city supervisor, to have "general charge of all matters pertaining to the administration of the government" of Englewood, N. J., "under the direction and control of the mayor and council" of the city, is provided for by an ordinance passed Dec. 20, effective Jan. 1, 1927. Announcement of the appointee is expected immediately. The salary will be $7,500 a year. Englewood is a suburb of New York City. It had a population of 11,627 in 1920. Douglas G. Thomson is mayor and F. Hamilton Reeve is city solicitor of Englewood.

Business Notes

F. E. SWINEFORD, chief engineer of the bureau of maintenance of the Ohio Department of Highways, has resigned to accept the position of service director of the city of Akron, Ohio. Mr. Swineford's resignation was accepted Dec. 12 and his appointment to the new position was of Jan. 1. Mr. Swineford's position in the Ohio Department of Highways will be filled by G. C. SNYDER, present division engineer of District No. 6, with headquarters at Marion. Mr. Snyder first entered the employ of the highway department as assistant resident engineer in 1912, later serving as assistant resident engineer and resident engineer of Marion County.

E. K. CORTRIGHT and H. B. MISH have formed a consulting engineering partnership to continue the business of BENJAMIN H. DAVIS, consulting engineer of New York, who recently died. The business will be conducted under the name of the B. H. Davis Company, consulting engineers, Whitehall Building, New York. Mr. Cortright had been associated with the late Mr. Davis for about 16 years and had been entrusted to direct many of the firm's most important projects.

W. E. FOWLER has been appointed general superintendent of the Montour Railroad Company, Pittsburgh, Pa., with supervision over transportation and maintenance.

R. G. CONE, until recently manager of Frank J. Ginder, Inc., paving and concrete construction contractors in Philadelphia, has been made resident engineer on the construction of the new Detroit River bridge, being employed by Modjeski and Chase, consulting engineers for the Detroit International Bridge Company. Prior to his being employed by the Ginder company, Mr. Cone was with the Delaware River Bridge Joint Commission as assistant engineer on foundations and later resident engineer on the construction of the main bridge.

COL. JAMES A. WOODRUFF, Corps of Engineers, has recently been appointed district engineer of the Second New York and the Porto Rico engineer districts, with headquarters at 39 Whitehall St., New York City. He has just returned from a three-year tour in the Canal Zone, where he was Chief of Staff of the Panama Canal Department.

HARRY K. ELLIS, for the past eight years county engineer of Chester County, Pennsylvania, has accepted a position in the designing department of the Phoenix Bridge Company.

J. E. ELLIOTT, engineer of the Trenton plant of the American Bridge Company, will become engineer of the Ambridge plant, Pittsburgh Division, effective Jan. 1. Mr. Elliott replaces R. G. Manning, who resigned. R. R. Graham, designing engineer of the New York office, will become engineer of the Trenton plant.

Engineering Societies

Calendar

Annual Meetings

INTERNATIONAL ASSOCIATION OF STREET SANITATION OFFICIALS, Chicago, Ill.; Annual Conference, Detroit, Mich., Jan. 9-10, 1928.

AMERICAN ROAD BUILDERS ASSOCIATION, Washington, D. C.; Annual Convention, Cleveland, Ohio, Jan. 9-14, 1928.

AMERICAN SOCIETY OF CIVIL ENGINEERS, New York City; Annual Meeting, New York City, Jan. 18-20, 1928.

ASSOCIATED GENERAL CONTRACTORS, Washington, D. C.; Annual Convention, West Baden, Ind., Jan. 23-27, 1928.

NATIONAL ASSOCIATION OF BUILDERS' EXCHANGES, Washington, D. C.; Annual meeting, Miami, Fla., Feb. 6-9, 1928.

ENGINEERING INSTITUTE OF CANADA, Montreal; Annual Meeting, Monday, Feb. 14-16, 1928.

AMERICAN CONCRETE INSTITUTE, Detroit, Mich.; Annual Convention, Philadelphia, Feb. 28, 29 and March 1, 1928.

AMERICAN RAILWAY ENGINEERING ASSOCIATION, Chicago, Ill.; Annual Convention, Chicago, March 6-8.

THE WESTERN SOCIETY OF ENGINEERS at its meeting on Dec. 12 had a series of papers on the topographic mapping of the United States, as follows: "The Need of Facts," Dr. George O. Smith, director of the U. S. Geological Survey; "Progress of Mapping the United States," Col. C. H. Birdseye, chief topographic engineer, U. S. Geological Survey; "Procedure in Mapping Chicago and Vicinity," O. H. Nelson, topographic engineer, U. S. Geological Survey; "Illinois Co-operation," Major W. H. Herron, Illinois State Geological Survey; "The Utility of Topographic Maps," M. M. Leighton, chief of Illinois State Geological Survey. Examples of these maps were on exhibition at the society's rooms. For Dec. 19, the program was a lecture on "Measurement of the Velocity of Light," by Professor A. A. Michelson.

THE THIRD MICHIGAN ENGINEERING CONFERENCE will be held at the Hotel Statler, Detroit, Jan. 27 and 28. The principal papers which so far have been arranged for are: "Relation of the Engineer to Management," John Lyle Harrington; "Electrification of Steam Railroads," W. M. Vandersluis, Illinois Central Railroad; "Welding," Prof. Comfort A. Adams, professor of engineering, Harvard University; "Mississippi River Problem," Arthur E. Morgan, president of Antioch College; "X-Ray as an Engineering Tool," Geo. L. Clark, University of Illinois; "Commercial Aviation," Wm. B. Stout, Stout Metal Airplane Co., airplane division, Ford Motor Company. Twenty-three engineering groups are sponsoring this conference.

Construction Equipment and Materials

A Section Devoted to What the Manufacturer Is Doing for the Engineer and Contractor

Western Road Show Set for Week of March 7, 1928

The Western Road and Equipment Exposition under the auspices of the Western Construction Equipment Distributors, will be held in Los Angeles, Calif., March 7-11, 1928. The display and demonstrations of machinery will be held in the Central Manufacturing District of Los Angeles, which has donated the use of a tract of 40 acres for the purpose.

European Cementation Process Introduced Into U. S.

In Europe and more particularly in Great Britain, a cementation process has been developed by the Francois Cementation Co. and used on many wet shaft sinking operations. The term cementation is applied to high pressure injections of liquid cement mixture into cavities, fissures, pores or cracks so that they are completely filled, rendering the rock structure impermeable to water. This process has recently been introduced and is now being employed in the United States by the Dravo Contracting Co., Pittsburgh, Pa. Major J. S. Crawhall has recently come from England to join the staff of the Dravo Contracting Co. as an expert on the cementation work.

It is reported that the Francois company is employed on such work in Europe as the restoration of St. Paul's Cathedral, the Mersey Tunnel, and the London tube subways. The Dravo Contracting Co. states in its announcement that while ordinary grouting has been and still is a very useful aid in engineering work, its application is limited, and that those engineers who have had long experience in grouting have been the first to recognize the possibilities of cementation in greater elasticity and efficiency.

An important feature of the Francois process is that a continuous stream of grout is pumped into the fissures at pressures ranging up to 3,000 pounds per sq.in. The fissures are rendered more tractable to the reception of grout by the lubricating action of a patented chemical process so that, it is claimed, the finest hair cracks are sealed. In mining, the method is principally applied to shaft sinking, tunnel driving through water-bearing or loose ground, sealing leakages in existing shafts and tunnels and in the construction of watertight mine dams. In the civil engineering field cementation is said to be even more extensive. Apart from tunnels and shafts in wet ground, the process is claimed to have proved sound and economical in preparing the underlying rock for reservoir construction and in avoiding deep core wall trenches. It has also been used in sealing leakages in existing reservoirs and culverts, in the conversion of gravels into concrete, and in the consolidation of defective foundations.

Business Notes

BYERS MACHINE CO., Ravenna, Ohio, has purchased the Massillon Power Shovel Co. The Byers company is taking over the entire responsibility of distribution and sales of Massillon shovels along with its Bear Cat line. The completed new organization will market a complete line of shovels and cranes from ½ yd. to 1¼ yd. capacity. The present manufacturing plants of both companies will be continued, Massillon shovels being manufactured as before at the Massillon, Ohio, plant.

HAZARD WIRE ROPE CO., Wilkes-Barre, Pa., has acquired the wire rope division of the Waterbury Co., New York City. The Hazard Co. had its inception as an iron wire and nail mill in Philadelphia in 1810 while the Waterbury Co. was founded in 1816 for the manufacture of fibre cordage.

GEORGE W. PAULETTE, formerly water superintendent at Emporia, Kansas, is now in charge of the International Filter Company's Kansas City office. Recently he represented E. W. Bacharach & Co. in the municipal filtration field.

YEOMANS BROTHERS CO., Chicago, announces the appointment of the Dempster Mill Mfg. Co., Amarillo, Tex., as its representative in northwestern Texas to handle the company's sewage ejectors and pumps.

New Developments

Chain Hoist Increased in Efficiency and Service

A new high speed chain hoist, known as Model K Cyclone hoist, has recently been manufactured by the Chisholm-Moore Mfg. Co., Cleveland, for which is claimed an increase in efficiency over the older model. Eight ball bearings, four roller bearings, and one Timken thrust roller bearing in the lower swivel hook contribute to this increase in efficiency. One of the most important changes was made in the four eccentric roller bearings, which in the old models were loose and had to be carefully fitted into their cage at the time of assembling the hoist. These roller bearings now are built in one complete unit which eliminates any loose rollers.

A special alloy steel has been employed on the lift wheel, where experience has shown that wear is first encountered. It is claimed that this new alloy will contribute to a much longer life for the hoist. The entire mechanism is inclosed in a dust-proof, oil-tight frame, lubrication needs being met by a packing with grease once a year.

Roller Bearings and Single Line Hoist Feature Power Shovel

A new one-yard shovel which can be converted to either a clamshell, dragline or pull shovel, operated by gasoline, Diesel or electric drive, at the purchaser's option, has recently been developed by the Ohio Power Shovel Co., Lima, Ohio. Important among the new machine's many features are the use of Timken roller bearings instead of the bronze sleeve or babbitt type and a single line hoist. It is claimed that the use of the roller bearings has contributed to great speed combined with more power and smoothness. Maintenance costs are also cut to a minimum. The roller bearings need lubricating about every six months and bearing adjust-

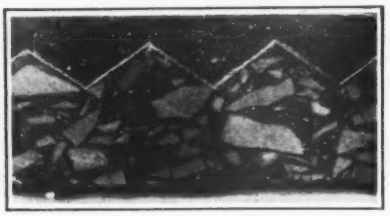
ments are claimed to be seldom necessary. It is claimed that one set of bearings will outlast the machine.

The single-line hoist which has long been used on large machines and allows a higher lift and greater reach, has been applied to this shovel; this the manufacturers claim, is its first application to a shovel of such small size.

Three-lever control is used, the position and throw being so arranged that any steam shovel operator can get maximum yardage in a very short time. Independent control of the dipper crowd is another feature. An all-gear propelling drive is used with all driving gears under the truck inclosed in oil-tight cases.

Asphalt Paving Method Ties Base to Wearing Surface

A special method of interlocking the base and wearing surface of asphalt pavement is being introduced into the United States by the Interlock Asphalt Paving Corporation, 2 Lafayette Street, New York City. A specially designed tamping-roller places transverse corrugation on the base course, thus providing key ways for the wearing course which is applied immediately following. A cross-section of the completed pavement is shown in the upper illustration, while the tamping roller is shown in operation spanning the pavement in the lower illustration.

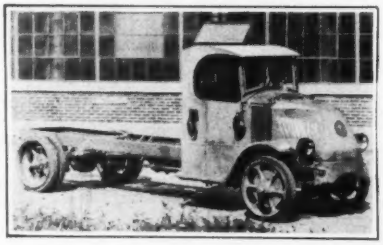


The interlocking feature is covered by patents issued by the Canadian Patent Office to Charles A. Mullen on March 17, 1925 and the method has had its widest application so far in the province of Ontario, Canada. The Interlock company has been granted the rights for the entire United States. Its plan is to supply the successful bidder with proper equipment and supervision wherever the pavement is specified.

It is claimed for the interlocked pavement that it insures a dense uniform structure impervious to moisture; that it overcomes the tendency of sheet asphalt pavement to shove; and that it gives a road of maximum stability and excellent riding quality.

New Motor Truck Announced in 3½-5 Ton Capacity

A new model motor truck known as Model AK with a capacity of 3½-5 tons and with 162-in., 174-in., and 186-in. standard wheelbases, has been announced by Mack Trucks, Inc., New York City. Many of the characteristics of the Mack "Bulldog" model are incorporated in this new model. These



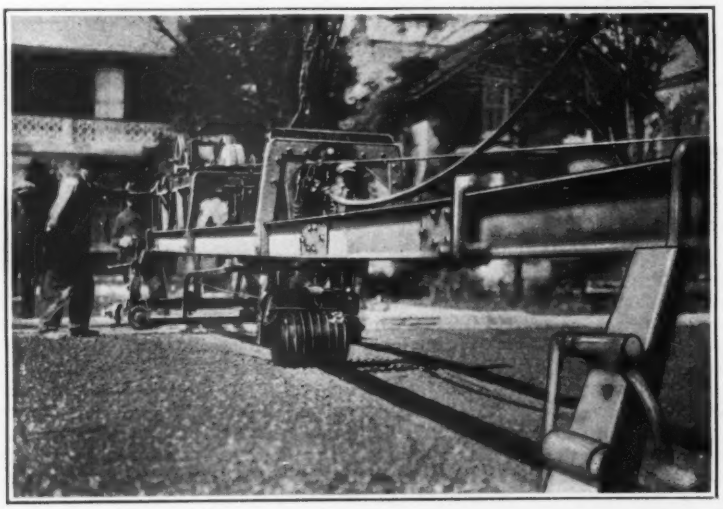
include heavy-duty four-cylinder engine with three-point suspension, heat-treated semi-steel cylinders, drop-forged and case-hardened crankshaft, force feed and splash lubrication, water pump of the balanced bronze rotor type, large single disk clutch, transmission with four speeds forward and one reverse. Among the special features is an engine of 4⅜-in. bore and 6-in. stroke with the cylinders cast en bloc with detachable aluminum heads. The transmission is suspended in four blocks of resilient rubber held under compression. Springs are rubber shock insulated, reducing the vibration to a minimum. In addition to shock insulating the cab, the seats themselves are mounted in rubber. Being a high-speed heavy-duty truck, four-wheel brakes of the internal expanding type have been made standard on this model. The emergency brake operates mechanically and is of the external contracting type. On the chain drive model it is located on both jackshafts while on the dual reduction model, it is mounted on the rear end of the transmission.

The new model has a sharper turning circle than on any previous Mack model. This has been effected by center point steering.

New Back Filler Designed for Tractor Attachment

A new front blade back filler known as the Miami-Bulldozer has recently been designed by the Miami Trailer-Scraper Co., Troy, Ohio, for attachment to the model 20-K Cletrac tractor. The blade is controlled by a Miami power winch attached to the power take-off on the rear of the tractor. This positive control of the blade enables the tractor driver to instantly raise or lower the blade if desired. The movement of the blade can be accomplished when the

tractor is either in forward or reverse motion. The unit is suitable not only for trench back filling work but may also be used in leveling work, for small grading jobs, for spreading gravel, crushed stone, etc., and for moving snow.



Manufacturers and Trade Associations

Calendar

Annual Meetings

NATIONAL SAND AND GRAVEL ASSOCIATION, Washington, D. C.; Annual Convention, Detroit, Jan. 4-6, 1928.

ASSOCIATED EQUIPMENT DISTRIBUTORS, Milwaukee; Annual meeting, Cleveland, Jan. 6-7, 1928.

AMERICAN ROAD BUILDERS ASSOCIATION, Washington, D. C.; Annual Convention, Cleveland, Ohio, Jan. 9-14, 1928.

NATIONAL CRUSHED STONE ASSOCIATION, Washington, D. C.; Annual Convention, West Baden, Ind., Jan. 16-19, 1928.

NATIONAL PAVING BRICK MANUFACTURERS ASSN., Chicago; Annual meeting, Chicago, Jan. 25-26, 1928.

Business Side of Construction

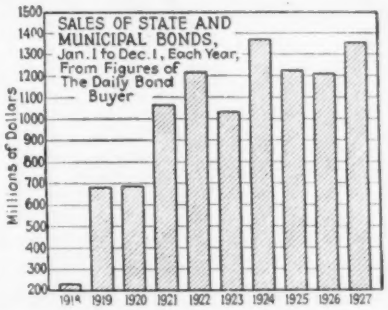
Facts and Events that Affect Cost and Volume

Record Total For 1927 Indicated In Public Improvement Bond Issues

THAT state and municipal financing in 1927 will aggregate close to $1,500,000,000 and establish a new high record for that class of borrowing, is estimated by *The Daily Bond Buyer* of New York.

The accompanying diagram shows that bond sales for the first eleven months of 1927 are well ahead of the corresponding period in any other years since 1918, with the exception of the record year 1924.

The following list of representative public bond issues, sold during the period Nov. 1 to Dec. 3, 1927, indicates that nearly 80 per cent of current disposals are selling above par. Figures in the table are compiled from data furnished by the *Commercial and Financial Chronicle*.



Demand for municipal bonds has been noticeably active since September. The quantity available for sale, however, is small, compared with the number of prospective buyers in the current market. Prices are, accordingly, firmer.

The December total is usually large for the reason that it contains, as a rule, many issues sold in earlier months but not promptly reported. Municipal borrowing, however, has been slower during the last few weeks than at any time since the beginning of the second quarter.

BOND SALES FOR STATE AND MUNICIPAL IMPROVEMENT—NOV. 1 TO DEC. 3, 1927

State	Purpose	Amount	Rate per Cent	Sold For	Basis	Dated	Maturity	Purchased by
Arkansas		$3,500,000	4¼	100.43	4.21		1934–51	Halsey, Stuart & Co.
Virginia		1,425,000	4	103.349	3.83	July 1, 1927	July 1, 1962	American National Bank of Richmond, and others.
County								
Alameda, Calif.	Oakland Inner Harbor tube	250,000	5	109.843	4.09	June 15, 1923	1941–42	Anglo-London-Paris Co. of San Francisco, and others
Allen, O.	Road	19,306	5½	104.15	....	Sept. 1, 1927	1928–37	W. L. Slayton & Co. of Toledo.
Decatur, Ga.	Paving	30,000	5	101.85	....			Bell, Speas & Co. of Atlanta.
Des Moines, Ia.	Road	100,000	4½	100.802	4.14	Dec. 1, 1921	1933–42	Harris Trust & Savings Bank of Chicago.
Fort Bend, Tex.	Road	675,000	5½	100.	....			H. C. Burk & Co. of Houston and others.
Fountain, Ind.	Highway	15,800	4½	102.75	4.02	Nov. 15, 1927	1929–38	Cities Securities Corp. of Indianapolis.
Glades, Fla.	Courthouse	50,000	6	100.18	5.98		1937–47	Hanchett Bond Co. of Chicago.
Highland, O.	Highway	35,195	5	100.65	4.33	July 1, 1927	1928–36	Otis & Co. of Cleveland.
Jefferson, Pa.	Road	140,000	4½	103.89	3.93	Dec. 1, 1927	1932–55	Jefferson Co. National Bank of Brookville.
Lewis, Tenn.	School Building	50,000	5½	100.	....	July 5, 1927	1947	Caldwell & Co. of Nashville.
London, Tenn.	School Building	50,000	5	100.40	4.93		1928–37	Caldwell & Co. of Nashville.
Pottawattamie, Ia.	Road	200,000	4½	100.7625	4.16	Dec. 1, 1927	1933–42	Geo. M. Bechtel & Co. of Davenport.
Shelby, Ia.	Bridge	148,000	4½	100.186	4.20	Oct. 1, 1927	1929–33	Carleton D. Beh Co. of Des Moines.
Taylor, Tex.	Road	600,000	4½	100.471	....			Mercantile Trust & Savings Bank of Dallas.
Union, Fla.	Road	350,000	6	100.	....	July 1, 1927	1931–52	C. A. Steed & Son of West Palm Beach.
Union, O.	Road	124,900	4½	100.05	4.21	Jan. 1, 1928	1929–33	First National Co. of Detroit.
Municipality								
Albuquerque, N. M.	Water, Sewer, etc.	590,000	4½	101.11	4.39		1931–46	Continental & Commercial Co., of Chicago, and others.
Armiston, Ala.	Municipal impvt.	35,000	5½	101.579	5.16	Dec. 1, 1927	1928–37	Ward, Sterne & Co. of Birmingham.
Benicia Reclamation Dist., Calif.	Permanent impvt.	125,000	6	90.	7.08	July 1, 1927	1937–47	Elmer J. Kennedy Co. of Los Angeles.
Burbank, Calif.	Sewer	325,000	4½	100.61	4.20	Nov. 1, 1927	1928–63	Anglo-London-Paris Co. of San Francisco.
Cambridge, Mass.	Building, Bridge, etc.	350,500	3¾	101.33	3.58	Dec. 1, 1927	1928–47	F. S. Moseley & Co. of Boston.
Castleton Fire Dist. No. 1, Vt.	Water	90,000	4½	100.	....	Dec. 1, 1927	1947	Allen National Bank of Fairhaven.
Clifton Forge, Va.	Waterworks	110,000	4½	100.	....	Dec. 1, 1927	1942	Prudden & Co. of Toledo.
Clifton Heights, Pa.	Highway	90,000	4½	102.70	4.095	Dec. 1, 1927	1957	E. H. Rollins & Sons of Philadelphia.
Cumberland, Md.	Hospital	150,000	4½	107.62	4.06	Dec. 1, 1927	1957	Harris, Forbes & Co. of New York.
Dayton, O.	Bridge	75,000	....	102.69	....	Dec. 1, 1927	1929–53	A. C. Allyn & Co. of Chicago.
Duluth, Minn.	City Hall	250,000	4½	102.954	3.94	Nov. 1, 1927	1928–52	Blair & Co. of New York.
East Orange, N. J.	General impvt.	523,000	4½	102.14	4.02	Nov. 1, 1927	1928–51	Phelps, Fenn & Co. of New York, and others.
	School	268,000	4½	101.97	4.04	Nov. 1, 1927	1928–48	
Eldorado, Kan.	Paving	17,100	4½	100.	....	Nov. 1, 1927	1928–37	School Fund Commission.
Eugene, Ore.	Municipal impvt.	154,637	5½	100.27	5.48		1937	Ferris & Hardgrove of Portland.
Forest Grove, Ore.	Water System	56,000	5	101.83	....	Nov. 1, 1927	1929–47	Atkinson, Jones & Co. of Portland.
Greensburg-Fairview Water Dist., N. Y.	Water	24,500	4.10	100.19	4.07		1928–40	Sherwood & Merrifield, Inc., of New York.
Hancock, Ind.	Township impvt.	37,000	4½	101.97	4.14	Dec. 1, 1927	1929–38	William A. Hughes.
Holyoke, Mass.	Gas and Electric Light	575,000	3½	101.53	3.57	Dec. 1, 1927	1928–47	Old Colony Corp. of Boston.
Kenmore, N. Y.	Paving and Sewer	245,000	5	102.13	4.32		1928–41	First National Bank of Kenmore.
Lackawanna, N. Y.	General impvt.	812,000	4½	100.86	4.14		1928–54	H. L. Allen & Co., and others.
Liberty, N. C.	Street impvt.	47,000	5½	100.531	5.42	Dec. 1, 1927	1930–47	Hanchett Bond Co. of Chicago.
Lincoln Park Dist., Chicago, Ill.	Park	1,000,000	4½	102.36	3.96	Nov. 1, 1927	1928–47	Harris Trust & Savings Bank of Chicago, and others.
Madison, Wis.	Sewer, Street, etc.	635,000	4½	103.633	4.06	Dec. 1, 1927	1927–47	First Trust & Savings Bank of Chicago.
Mangum, Okla.	Waterworks	75,000	4½	100.02	....			Liberty National Bank of Oklahoma City.
	Sewer	49,000	4½	100.02	....			
Mount Airy, N. C.	Water	125,000	5	100.	....	Oct. 1, 1927	1932–57	Taylor Wilson & Co., Inc. of Cincinnati.
Oakland, Mich.	Road	602,000	4½	100.46	4.41		1930–38	Watling, Lerchen & Hayes of Detroit.
		217,000	4½	100.004	4.49		1930–37	Howe, Snow & Co. of Detroit.
Oaklyn, N. J.	Municipal impvt.	40,000	5½	100.	....	Nov. 1, 1927	1933–37	M. M. Freeman & Co. of Philadelphia.
Orlando, Fla.	Paving	155,000	5	100.69	4.86	Oct. 1, 1927	1928–38	W. L. Slayton & Co. of Toledo.
Pasadena, Calif.	Sewer, etc.	65,000	4½	104.23	4.24		1933–39	Drake, Riley & Thomas of Los Angeles.
Pawtucket, R. I.	Waterworks	300,000	4	100.15	3.99	Nov. 1, 1927	1932–52	Bankers Trust Co. of New York.
River Rouge, Mich.	Basement	75,000	4½	101.10	4.27	Nov. 1, 1927	1928–37	Detroit Trust Co. of Detroit.
Rockville Center, N. Y.	Incinerator	100,000	4.10	100.33	4.01	Dec. 1, 1927	1928–53	Graham, Parsons & Co. of New York.
	Water	5,000						
Russell, Ia.	Municipal impvt.	4,500	4½	101	4.34	Dec. 1, 1927	1929–40	Citizens State Bank of Russell.
Saint Joseph, Mo.	Bridge	304,000	4½	103.97	4.10	Nov. 1, 1927	1932–47	Harris Trust & Savings Bank of Chicago.
Salem, Ore.	Paving	75,340	6	105.67	5.27	Nov. 1, 1927	1937	Atkinson, Jones & Co., Inc., of Portland.
Schenectady, N. Y.	Water, Sewer, etc.	478,000	4	100.94	3.86	Oct. 1, 1928	1928–47	Dewey, Bacon & Co of New York.
Stanford, Mont.	Water and Sewer	60,000	6	100	....	Jan. 1, 1928	1948	State of Montana.
Walla Walla, Wash.	City Hall	65,000	4.20	100	....	Jan. 1, 1928	1930–43	State of Washington.
Waterville, O.	Waterworks	36,000	5	100.97	....	Oct. 1, 1927	1928–37	Waterville State Savings Bank.
Wyandotte, Mich.	Municipal impvt.	129,900	4½	100.52	....			Braun, Bosworth & Co. of Toledo.
Zelienople, Pa.	Sewage Disposal	16,000	4½	104.12	4.10	Nov. 1, 1927	1934–49	E. H. Rollins & Sons of Philadelphia.

This Week's Contracts—With Comparisons

Minimum costs observed are: $15,000 for water-works and excavation, drainage, irrigation, levee, river and harbor projects; $25,000 for other public works; $40,000 for industrial and $150,000 for commercial, educational, institutional, religious and other buildings.

Week ended	Public Work	Private Work	Total U. S.
December 22, 1927.	$23,599,000	$41,777,000	$65,376,000
December 15, 1927.	13,515,000	46,115,000	59,630,000
December 23, 1926.	19,081,000	14,931,000	34,011,000
Jan. 1 to date			
1927	1,236,955,000	1,936,362,000	3,173,317,000
1926	1,076,824,000	1,748,997,000	2,825,821,000

E.N.-R. Index Numbers

On December 1, 1927

COST	VOLUME
203.90	272
Last Year 210.80	Last Year 211

The Index Numbers are published in detail and with charts in the first issue of each month. The Cost Index is given in full from 1903 through 1926, with a thorough explanation of its method of computation, and a graphic comparison with many other index numbers, in the 48-page pamphlet entitled Engineering News-Record Construction Costs.

Weekly Construction Market

	New York	Atlanta	Dallas	Chicago	Minneapolis	Denver	San Francisco	Seattle	Montreal
Steel Products									
Structural shapes, per 100 lb.	$3.34	$3.75	$4.00	$3.10	$3.35	$3.95	$3.00	$3.00	$4.00
Structural rivets, per 100 lb.	5.00	4.00	4.75	3.60	3.85	4.64	5.75	5.25	5.50
Reinforcing bars, per 100 lb.	3.24	2.99	2.75	2.50	2.87½	3.87½	2.95	3.00	3.57
Steel pipe, black, 2½ to 6 in., lap weld, discount from list	48%	40%	54%	51%	48%	41%	42@53.8%	48%	.....
Cast-iron pipe, 6 in. and over, per net ton	38.60	+36.50	52.00	34.60	40.00	52.00	48.50	53.00	41.00
Concreting Material									
Cement, without bags, per bbl	2.25@2.35†	2.45	2.05	2.05	2.22	2.85	2.51	2.65	1.41
Gravel, ¾ in., per cu.yd.	1.75	.....	2.38	2.00	1.25*	1.90	1.80	—1.00	1.50*
Sand, per cu.yd.	1.00	1.75	2.00	2.00	1.25†	1.00	1.40	—1.00	1.25*
Crushed stone, ¾ in., per cu.yd.	1.85	.....	—2.75	2.00	1.25*	2.50	1.70	—1.90	2.00*†
Miscellaneous									
Fir, 3x12 to 12x12, 20 ft. and under, per M.ft. b.m.	.....	.....	.....	39.50(848)	36.75 @38.75	33.75	27.00	24.00	50@55
Pine, 3x12 to 12x12, 20 ft. and under, per M.ft. b.m.	60@62†	34.00	56.00†	.....	.....	.....	.....	.....	.....
Lime, finishing, hydrated, ton	22.50@24.50†	21.50	19.00	20.00†	25.50	24.00	27.50	24.00	21.00
Lime, common, lump, per bbl	3.25@3.50†‡	1.25	1.82	1.50†	1.70	2.70‡	1.70	2.80‡	10.00*
Common brick, per M	16.00@17.00†	11.50	13.50	12.00†	13.75†	8@12	13.00	14.00	20.25
Hollow building tile, 4x12x12, per block	.....	.068	.094	.....	.072	.085	.....	.10	.10
Hollow partition tile, 4x12x12, per block	.0966	.068	.094	.076†	.072	.085	.108	.09	.08
Linseed oil, raw, 5 bbl. lots, per 7½-lb. gal.	.79½	—.81¾	.85	—.76	— 84	.85	84	.98	.90
Common Labor									
Common labor, union, per hour.	.90⅜	.30	.....	.90	.....	.....	.50@.60	.70	.....
Common labor, non-union, hour.	.....	.25	.30@.50	.82½	.45@.60	.31¼@.50	....	.62½	.30@.35

*Ton. †Delivered to job. ‡280-lb. net. +Advance. —Decline.

Explanation of Prices

Structural Shapes: Beams, 3 to 15-in.; channels, 3 to 15-in.; angles 3 to 6-in., ¼-in. thick; tees, 3-in. and larger. Prices are per 100-lb., at warehouse, in less-than-carload lots.

Structural Rivets: Round and cone head, per 100-lb., in full keg lots, at warehouse.

Reinforcing Bars: Rolled from billets, ¼-in. and larger, per 100-lb., out of stock at warehouse.

Wrought-Steel Pipe: Black, 2½ to 6-in., lap welded, prevailing discounts from standard list, at warehouse, in less-than-carload lots.

Cast-Iron Pipe: Class B and heavier, size 6-in. and over, per net ton, in carload lots, f.o.b. city.

Portland Cement: Per bbl. of four bags, without charge for containers, in carload lots, f.o.b. city, or delivered to job, according to notation in table.

Gravel: Size ¾-in., per cu.yd., in carload lots, f.o.b. city. Weight per cu.yd.: New York, Atlanta, Dallas, Chicago, Montreal, 3,000 lb.; Seattle, 3,200 lb.; Minneapolis, San Francisco, 2,800 lb.; Denver, 2,700.

Sand: Per cu.yd., weighing 1½ tons (Seattle 2,850 lb.) in carload or cargo lots, f.o.b. city.

Crushed Stone: Size ¾-in., per cu.yd., in carload lots, f.o.b. city. Weight per cu.yd.: New York, Atlanta, Dallas, Chicago, Montreal, 2,500 lb.; San Francisco, 2,400 lb.; Minneapolis, 2,600 lb.; Denver, Seattle, 2,700 lb.

Pine and Fir Lumber: Prices are for rough timbers, size 3 x 12-in. to 12 x 12-in., in 20 ft. lengths and under, per M. ft. b.m., in carload lots to contractors, f.o.b. city. Long-leaf yellow pine is quoted in certain of the cities and Douglas fir in the rest of those covered by the table, according to nearness to source of supply.

Lime: Hydrated, finishing lime, per ton, at warehouse, in carload lots.

Common lump lime, at warehouse, in carload lots, per small bbl. of 180-lb., net, or large, 280-lb., net, as noted in the table.

Common Brick: Price per M. in carload lots, f.o.b. city, or delivered to job, as noted in the table.

Hollow Tile: Building (load bearing) or partition tile, 4 x 12 x 12-in., per block, in carload lots, f.o.b. city, or delivered to job, as noted in the table.

Linseed Oil: Prices are for raw oil in 5-bbl. lots, per 7½-lb. gal., delivered to job.

Common or Unskilled Labor: Rates are per hr., for pick-and-shovel or clean-up men in construction operations.